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TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. LXXVI

DECEMBER, 1913

Edited by the Secretary, under the direction of the Committee on Publications.
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NEW YORK

PUBLISHED BY THE SOCIETY

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ERRATA.

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Page 1060, line 24, for "drawn" read "thrown."
Page 1077, line 15, for "Le Baron" read "Le Conte."
Page 1078, 9th line from bottom, for "Le Baron" read "Le Conte."
Page 1172, last line, for "California" read "Mexico."
Page 1173, line 4, for "Mexico" read "California."
Page 1181, line 5, for "Miss Ella M. Blackman" read "Miss Ella M. Blackmar."

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Page 299, line 28: Substitute f for p, the sentence to read: "in stating that $f - p$ should be used for f in the derivation of the formula," etc.

Page 1092, 3d line below Fig. 7: For $Q_1 = ch_1 (h_1)^{\frac{3}{2}}$ read $Q_1 = cb_1 (h_1)^{\frac{3}{2}}$.

Page 1092, 4th line below Fig. 7: For $Q_2 = ch_2 (h_2)^{\frac{3}{2}}$ read $Q_2 = cb_2 (h_2)^{\frac{3}{2}}$.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1239

THE SIXTH AVENUE SUBWAY OF THE HUDSON AND MANHATTAN RAILROAD.*

By H. G. Burrowes, M. Am. Soc. C. E.

With Discussion by Messrs. T. B. Whitney, Jr., William J. Boucher, Lazarus White, H. L. Oestreich, and H. G. Burrowes.

Introductory.

The Hudson and Manhattan Railroad work described in this paper is that portion built under Sixth Avenue between 12th and 33d Streets, New York City, as shown on Plate I. The general plan consists of a two-track, reinforced concrete subway in the center of Sixth Avenue from the end of the iron-lined tunnels at 12th Street north to and including a three-track station at 33d Street. The extension northward from this Station to the Grand Central Station was authorized by a later franchise, and is not considered in this paper.

There are five stations. Those at 14th, 19th, 23d, and 28th Streets are similar to one another in type and practically the same in detail, with the exception of the entrances. The 33d Street Station is more comprehensive in all its details, and the structure and its general arrangements and equipment call for particular attention, it being of a new type compared with the usual stations on New York City tunnels.

In prosecuting this work, it was necessary to support temporarily the entire elevated railway, the entire surface railway, and also all

* Presented at the meeting of October 2d, 1912.

the pipes and electric cable properties between 12th and 33d Streets. Before making the excavation, it was necessary to construct temporary channels for all existing sewers and then build new sewers, one on each side of the structure; support all vaults within the limits of the work; remove and replace all vaults, and support all buildings between 31st and 32d Streets, carrying their foundations down to sub-grade; replace the elevated and surface railways on permanent concrete and brick foundations resting on the roof of the structure; relay all pipes and electric subways in the street, supporting them on timber bents; replace the electric cables in new ducts; restore the water, gas, and electric services; back-fill over the subway; and relay the asphalt pavement.

Under the conditions of the franchise, the street surface had to be maintained for the unobstructed use of traffic. This made it necessary to bridge over the entire surface and maintain the planking in first-class condition. The franchise also required that no openings be made in the street planking other than those absolutely necessary for the economic prosecution of the work.

The subway and stations throughout are of reinforced concrete, and the columns in the stations are of steel or cast iron.

Standard Tunnel Section.—The standard tunnel section, shown on Plate II, was used between the stations and in the work between 27th and 33d Streets, but, between 12th and 27th Streets, the walls are 4 in. thicker. At the track cross-over enlargement, between the 19th and 23d Street Stations, the section was modified by the elimination of the center wall and by changing the roof from an arch to an I-beam, concrete construction, supported on the side-walls alone. There was also a modification between 12th and 13th Streets due to the widening of each tunnel in order to provide for the reverse curve in the tracks made necessary by the difference in track centers in the standard subway and in the iron-lined tunnel below 12th Street, the distance between the tracks being 13 ft. 3 in. in the standard subways and 17 ft. 6 in. in the iron-lined tunnels.

Standard Stations.—As shown on Plates II and III, the standard stations differ little in detail except in the entrances. These are built under the stairways of the elevated railway, with the exception of the east entrance of the 23d Street Station, which is through the store on the southeast corner; the east entrance of the 19th Street Station, which is through the store on the southeast corner; the west

PLATE 1.
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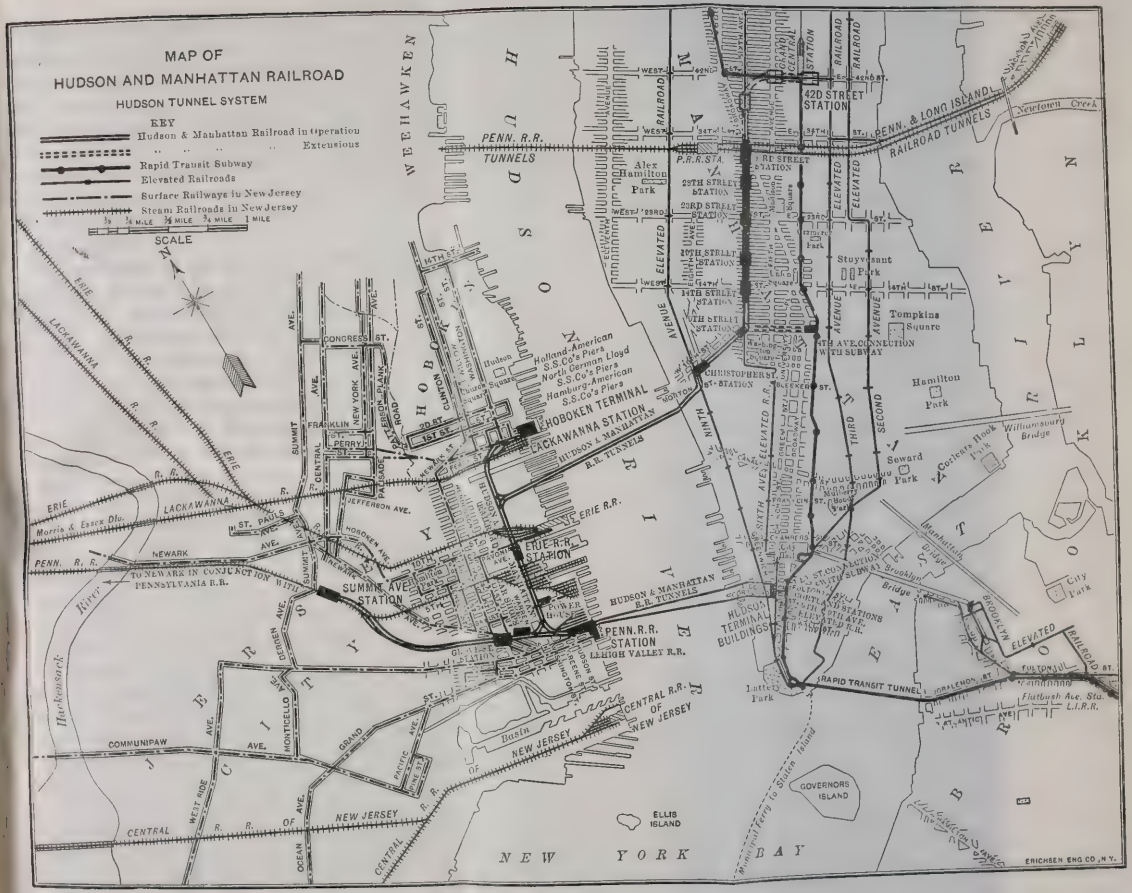
BURROWES ON
SIXTH AVENUE SUBWAY,
H. & M. R. R.



entrance of the 19th Street Station, which is through the store on the northwest corner; and the east entrance of the 14th Street Station, which is through the store on the southeast corner. Where entrances were made through stores, or private property, an agreement was entered into with the owners granting a right of way through the building. This was considered to be of advantage to the owners, as many patrons of the subway would pass through the stores, with consequent increase in their trade. At the points where such entrances were established it was necessary to make alterations in the interiors of the buildings.

The platforms of the stations rest on the foundations of the columns, on the side-walls, and on a brick wall under the overhang of the platform. At the 28th Street Station, however, a concrete girder was built, replacing the brick wall.

The roof of each station is of groined arches resting on the side-walls, the center-wall, and cast-iron columns.

Toilets are provided on each platform, and are drained by an automatic ejector operated by air power, which discharges into the sewers alongside the stations, with the exception of the 19th Street Station, where the sewer is below the level of the platform and the connection is made directly into it.

Floor drains were provided in the platforms and station entrances in the 14th and 19th Street Stations, but subsequent use has shown that they become clogged because so little water is passed through them in washing the platforms, and the dirt and sweepings remain in the drain heads. In the 23d and 28th Street Stations the platforms are sloped toward and drained to the track.

The cast-iron columns of the stations are 15 in. in diameter, 2 in. thick, and rest on bases having a bearing area of 2 304 sq. in. The area of the caps for the support of the concrete roof is 1 521 sq. in. The reinforcing rods are shown on Plate II, and need no description.

The Thirty-Third Street Station.—This is a three-track "through" station, as shown on Plate IV, and, for convenience in this description, will be taken to include the enlargement at the south end. Two enlargements were planned, one at each end of the station, to provide room for the track arrangements necessary to change from two tracks in the subway to three tracks in the station.

The station is built on rock foundation, and in general consists of

framed steel columns resting on concrete foundations, with side-walls and groined arch roofs and concourse floor. The roofs and the concourse floor are supported on the side-walls and on steel brackets riveted to the steel columns.

The columns of the elevated railway, excepting the two in the center of 32d Street, are supported on concrete piers based on steel grillages which span over, and distribute each column load on, four of the station columns. Each station column is in one piece, and passes through the concrete of the roof and the concourse floor. The elevated railway columns in the center of 32d Street are supported on steel towers made up of the extended columns of the station.

Over the enlargement, the elevated railway column foundations or piers rest directly on the roof, and under them extra girders are embedded in the roof for their support. These girders are supported by the side-walls and by steel columns in the center-walls.

The floor or invert of the station, between the foundations of the columns and the side-walls, is simply a concrete floor resting on the rock bed to carry the tracks, etc., and was planned to be 12 in. thick; but, owing to the unevenness of the rock surface as blasted out, it varies, sometimes reaching 3 ft.

All the station columns are 13 ft. 10 in. from center to center, and the groined arches, therefore, are in square panels and of the same radius, 8 ft. 3 in., in both directions.

The roof arches of the station are of reinforced concrete, 12 in. thick at the crown over the concourse, where the depth of fill was 10 ft., and 24 in. thick at the crown in the south end of the station, where the depth of fill varied from 18 to 20 ft.

There are five entrances to this station, as shown on Plate IV: three under the stairways of the elevated railway station at Sixth Avenue and 33d Street, covered with metal kiosks; one through the department store between 32d and 33d Streets, and one under Greeley Park, covered with an ornamental kiosk of artificial stone. In addition to these entrances which lead directly to the street, there are three stairways leading up to the street floor of the store between 32d and 33d Streets. There are, therefore, eight entrances, all leading to the concourse floor of the station.

The station is 387 ft. long on the train platforms, providing for the unloading of 8-car trains, and, throughout its entire length,

FIG. 1.—GENERAL VIEW OF STREET CONDITIONS WHILE RELAYING PIPES ON SIXTH AVENUE.

FIG. 2.—ROCK EXCAVATION IN ONE OF THE LARGE PITS.

occupies the full width of Sixth Avenue, being 94 ft. 10 in. wide between the interior faces of the side-walls.

There are two side platforms, 15 ft. 1½ in. wide, and two center platforms, 18 ft. 5 in. wide. Over the northern half of the train platform, between 32d and 33d Streets, there is a concourse, the full width of the station and 152 ft. long. Leading to this concourse there are twelve reinforced concrete stairways, three to each of the four platforms.

The roof is of groined arches of the same type as used in the standard stations, and the walls throughout are lined with ornamental glazed tile.

The framed steel columns of the station are encased in artificial stone slabs, made on the work, and the twelve stairways between the concourse floor and the train platforms have railings and balustrades of the same material.

At the north end of the concourse there are toilets, telephone booths, and four ticket offices for the various railroads served by the tunnel trains. At the south end there are two porter's rooms and a fan-room for the exhaust fan which discharges the foul air from the subway.

A signal tower and a battery-room are at the south ends of the two center platforms for the operation of the mechanism of the cross-over, etc., of the track in the enlargement. The same provision for tower and battery-room will be made at the north end of the station when the extension is built.

Under the Sixth Avenue front of the department store between 32d and 33d Streets on the west side of Sixth Avenue, and at the level of the platform, there is a large baggage and storage-room, 194 ft. long and 69 ft. wide, with access to the west platform, for the handling of baggage. This baggage-room has a large baggage chute and an escalator for raising and lowering parcels, etc., between it and the street level at 33d Street. On the 33d Street side of the same store building there is an entrance and handling-room, at the heads of the chute and escalator, so that baggage by truck, etc., may be received from and delivered at the street level.

Over the north end of this baggage-room and at the level of the concourse floor, there is a parcel check-room which is connected with

the large baggage-room below by a small chute and dumb-waiter for small parcels.

This station has four electrically-operated fans, two at the south end of the concourse, for the discharge of foul air from the subway, and two at the south end of the large baggage-room, for the supply of fresh air to the station. The exhaust fan draws the foul air through a reinforced concrete duct laid on the roof of the station and subway from a point north of the end of the 28th Street Station, which is the high point in the grade of the east or up-town tube. The fresh-air fan supplies the air through a concrete duct laid below the foundation of the station, at right angles to the track. This duct has openings under the platforms with louvers through which fresh air is supplied. The foul air is discharged through an opening in Greeley Park; the fresh air is taken in through an opening beside the department store at 32d Street.

Two automatic ejectors, one in the middle of the station, under the west center platform, and one at the north end of the same platform, are operated by compressed air. They are set in sumps which collect the drainage of the subway and station at this point. The ejector at the north end of the platform also collects and discharges the drainage of the toilets. These ejectors lift the drainage 30 ft., and discharge it directly into the sewers.

The concourse floor is of the same type as the roofs, the thickness being 6 in. at the crown of the arches.

The platforms are of 8-in. reinforced concrete slabs resting on 13-in. reinforced girders, the ends of which are carried on the concrete jackets around the bases of the steel columns and on the side-walls. These concrete girders and slabs were built in one operation. The tops of the platform and entrance floors are finished with a 1-in. coating of granolithic pavement which was laid as soon as the concrete had set sufficiently.

Structural Steel.—At 32d Street, where the station crosses the Pennsylvania Tunnel, the track level or grade line passes through the arch of the roof of that tunnel. It was necessary, therefore, to remove the reinforced concrete arch of the tunnel and replace it with heavy girders resting on the tunnel walls. The columns of the station and the track and platforms rest on these girders.

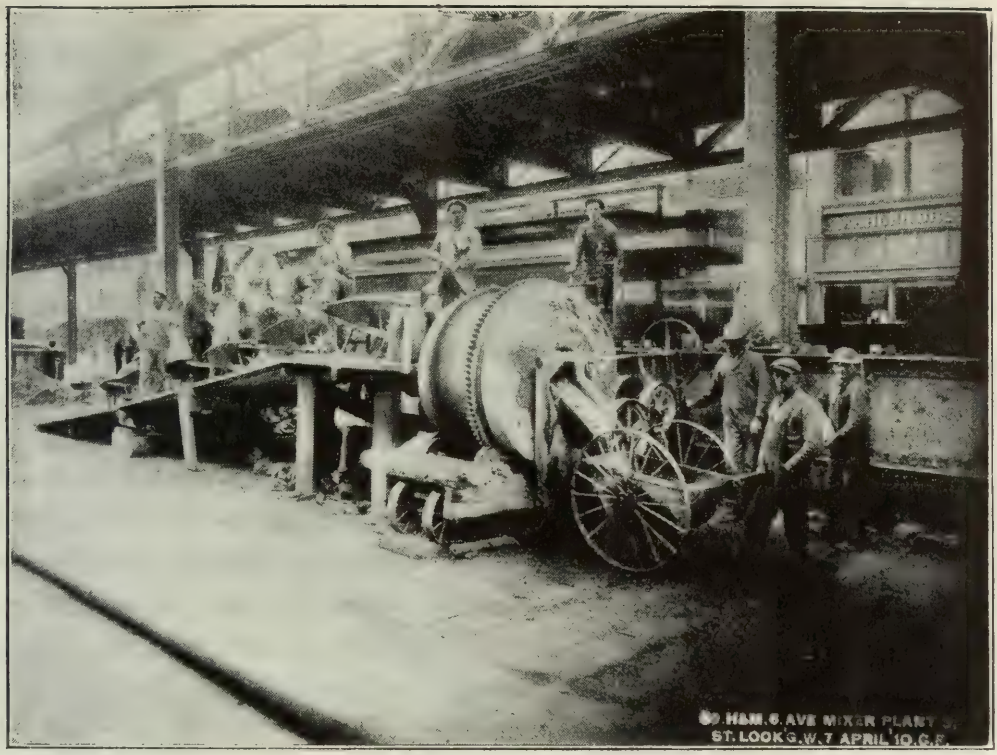


FIG. 3.—CONCRETE MIXER AND GANG.

FIG. 4.—CABLEWAY ON SIXTH AVENUE.

The steel columns of the station, as shown on Plates II and IV, are made up of two $\frac{5}{8}$ to $\frac{3}{4}$-in. plates and two 10-in., 30-lb. channels riveted to form box columns, and these have brackets at the springing line of the arches for their support. The bearing areas of the base castings vary from 961 to 1 296 sq. in., depending on the loads carried. The grillages on top of the station columns consist of three and sometimes four girders on which rest the four 24-in. I-beams which support the foundations of the elevated railway columns.

The twenty-four heavy girders spanning the walls of the Pennsylvania Tunnel are grouped in clusters of four under the station column lines; and the four I-beams are in two pairs near the side-walls. The six clusters of girders rest on steel billets placed between their ends and a grillage of 20-in. I-beams embedded in the walls of the Pennsylvania Tunnel, and support the twenty-four columns of the station which are over the tunnel, a steel billet being placed under each column spanning the four girders. The columns are anchored to the billets. Eight of these station columns form the two towers supporting the two elevated railway columns in the center of 32d Street. These eight columns pass through the concrete roof of the station.

The track is carried across the Pennsylvania Tunnel on a steel trough flooring resting on angle steps fastened to the webs of the outer girders of each group. The ties of the track are laid in this trough floor, as shown on Plate II.

Steel Reinforcing Rods.—In each panel of the roof over the concourse there are two sets or grids of rods laid so that the nearest rod is 2 in. below the top face of the concrete and 2 in. above the crown of the arch. The upper set of rods in each panel is composed of seven 1-in. square rods laid each way, transversely and longitudinally. The lower set is composed of fourteen 1-in. square rods laid each way in each panel. In each panel of the roof over the south end of the station, where the load is greater, the upper set of rods is 6 in. below the top surface of the concrete and consists of seven 1-in. square longitudinal and fourteen 1-in. square transverse rods per panel. The lower set is 2 in. above the crown of the arch and of the same number per panel both ways as in the upper set. In the concourse floor there is only one set of rods, and these are 2 in. above the crown of the arch, there being fourteen transverse and seven longitudinal rods per panel, and all are 1 in. square. Around the steel columns (where

they pass through the roof) there are additional special bent rods as shown on Plate II, in the form of a basket cluster. The reinforcing rods in the side-walls are 1 in. square, and 2 in. from the faces of the walls. The vertical inside rods are 12 in. from center to center and the horizontal rods 24 in. from center to center. The vertical outside rods are 9 in. from center to center and the horizontal rods 24 in. from center to center. The rods in the platform girders and slabs are 2 in. from the lower sides. There are seven $\frac{3}{4}$-in. square rods in the bottoms of each girder and $\frac{1}{2}$-in. square rods 8 in. apart each way in each slab or panel formed between the columns of the station.

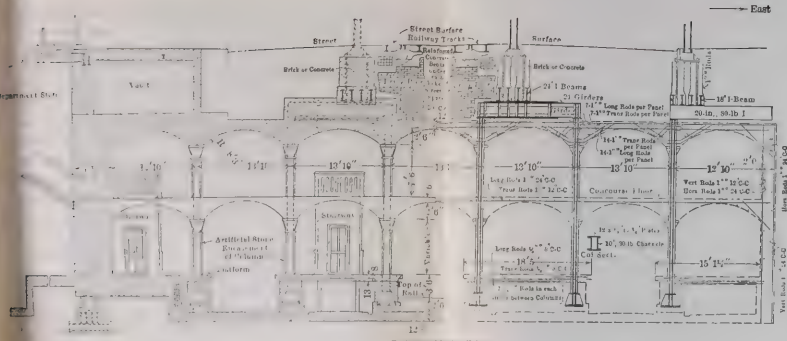
There are 1-in. square rods 2 in. from the outside faces of the 5 by 5-ft. elevated railway column piers, and 11 in. from center to center, to reinforce the piers, because it was impossible to increase the areas of the bottoms of the piers, owing to the position of the steel grillages supporting them. This reinforcement in the column piers was placed where the depth exceeded 6 ft.

Station Finish and Decoration.—After the completion of the station walls, floors, roofs, and platforms, the interiors were finished and decorated. The floors and platforms are finished with a granolithic top. The side-walls are tiled throughout, from the floors to the springing lines of the roof arches, with 6 by 6-in. white tile with green trim, laid in a mortar bed. All the stairways are equipped with wooden hand-rails, and the steps with iron safety treads embedded in the granolithic top finish. The ceilings and the faces of the center walls are finished with a wash of white cement and white sand or marble dust, applied with a brush. In the stations at 14th, 19th, and 23d Streets, the ceilings were first plastered to give a smooth appearance, but this was not done in the 28th and 33d Street Stations because it was decided that the form marks, or visible lines left by the lagging, were not detrimental to the appearance, as they conformed to the lines of the arches.

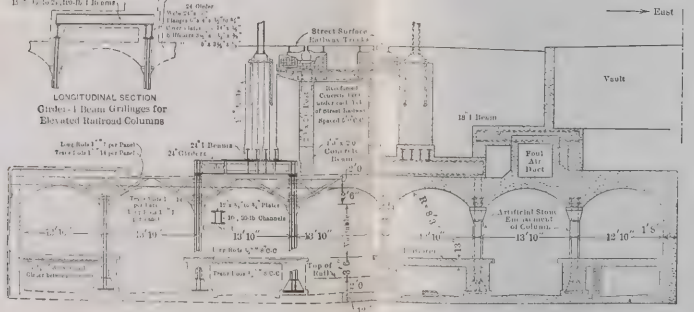
The iron railings at the ticket booths and the cast-iron columns are painted with an enamel green, conforming to the station color. The woodwork of the ticket booths, etc., is painted or varnished to conform to the color of the station.

The steel columns of the 33d Street Station are encased in moulded slabs made of white cement and marble dust, and the railings and

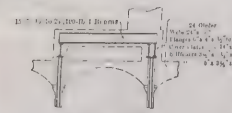
PLATE II
TRANS. AM. SOC. CIV ENGRS.
VOL. LXXVI, No. 1239.
BURROWES ON
SIXTH AVENUE SUBWAY
H & M. R. R.



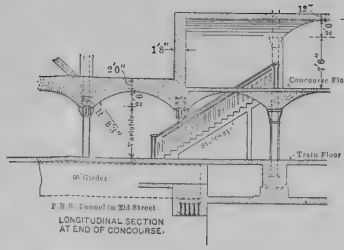
TRANSVERSE SECTION
North Half of 33d Street Station



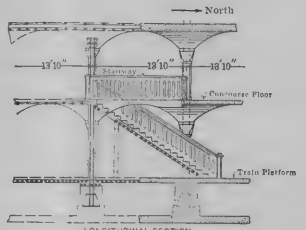
TRANSVERSE SECTION
South Half of 33d Street Station



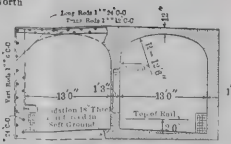
LONGITUDINAL SECTION
Girder - I Beam Gratings for
Elevated Railroad Columns



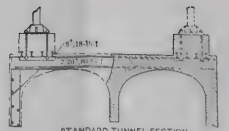
LONGITUDINAL SECTION
AT END OF CONCOURSE.



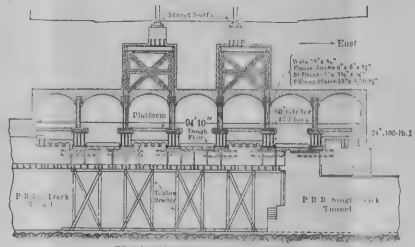
LONGITUDINAL SECTION
AT CONCOURSE.



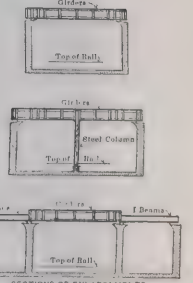
STANDARD TUNNEL SECTION
BETWEEN STATIONS.



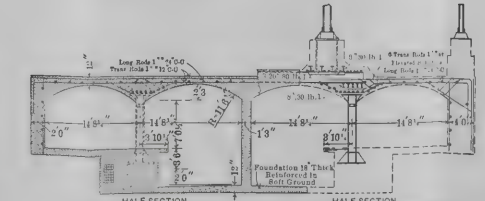
STANDARD TUNNEL SECTION
AT ELEVATED RAILROAD COLS.



TRANSVERSE SECTION AT 32D STREET.

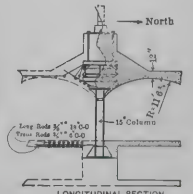


SECTIONS OF ENLARGEMENTS.

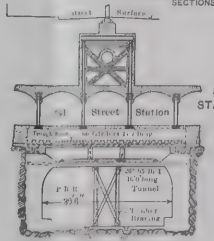


HALF SECTION
Standard Station-between Columns.

HALF SECTION
Standard Station-at Elevated Railroad Columns.



LONGITUDINAL SECTION.



LONGITUDINAL SECTION AT 32D STREET.

SECTIONS OF TUNNELS
STATION & ENLARGEMENTS

balustrades of the stairways between the train and concourse floor, are of the same material. All this artificial stone was made on the work.

Enlargement.—The enlargement south of the 33d Street Station is on rock foundation, and in general consists of one or two center-walls, excepting at the track cross-over, and two side-walls, all resting on rock. These walls support a flat roof made up of steel girders embedded in concrete on which rest the elevated railway, the surface railway, the street, etc. This enlargement is 280 ft. long, between the standard tunnel and the 33d Street Station, 27 ft. 3 in. wide at the south end and 92 ft. 4 in. wide at the north end. The thickness of the side-walls is 18 in. at the south end and 24 in. at the north end. The center wall is 15 in. thick, embedding the steel columns supporting the roof girders.

Street Surface Car Tracks.—The surface car tracks, shown on Plates II, V, and VI, are supported by brick piers and reinforced concrete bents under each yoke of the car track, 5 ft. from center to center. Over the concourse, and where the depth between the yokes of the street car tracks and the top of the roof is less than 7 ft., the piers are of brick and rest on the roof of the tunnel and stations. These piers are built transversely across the street car tracks. Over the south half of the 33d Street Station and over the enlargement and tunnel, where the depth between the car track yokes and the top of roof is from 7 to 17 ft., there are reinforced concrete bents. Two plain concrete girders, 24 in. wide and 4 ft. deep, are set longitudinally in order to distribute the loads of the reinforced concrete bents. These bents were cheaper than brick piers for depths exceeding 7 ft. The legs of these bents rest on the plain concrete girder built on the roof.

Buildings.—The architects of the store building on the west side of Sixth Avenue between 32d and 33d Streets, having the construction of the subway in mind, carried the column foundations of that building down to the sub-grade of the tunnel, so that it was not necessary to underpin this building. Between 31st and 32d Streets on both sides of Sixth Avenue it was necessary to support the buildings temporarily and carry their foundations down to rock or to sub-grade.

Drainage.—In addition to the two sumps in the 33d Street Station, there are two others to provide for the drainage of the subways.

These are equipped with automatic ejectors which discharge into the sewers.

Air Line.—An air line in each tube supplies compressed air for operating the signals, pumps, or ejectors, and for the use of air tools and in making necessary track repairs. These air lines are of wrought-iron pipe, and are reduced in diameter from 8 in. at 12th Street to 3 in. at 33d Street.

Subway Ducts.—There is a duct bank in each tube between 12th Street and the 33d Street Station. There are splicing chambers at each end of each station in both tubes, and also between stations, where necessary to shorten the length of cable haul.

Water-Proofing.—The subway and stations between 12th and 27th Streets are water-proofed with the regular burlap and pitch, and between 27th and 33d Streets with a compound or powder mixed with the concrete.

General Methods.

From 12th to 27th Streets.—The first step, as shown on Plate V, was to excavate pits between the columns of the elevated railway to or near sub-grade, and, in them, build up the timber bents to carry the girders which supported the elevated railway columns.

Between 12th and 18th Streets (excepting at 14th Street and because of wet foundation), pits were dug to within 8 or 12 ft. of sub-grade, between the elevated railway bents to be supported in the first operation, then timber piles were driven in the bottoms to hardpan or rock, and timber bents were built and framed on the piles to support the girders. These bents were carried above the street surface, and carried the I-beams or girders supporting the elevated railway columns.

Between 18th and 20th Streets, the pits between the elevated railway bents were carried down to sub-grade, the subsoil being dry. Between 20th and 27th Streets, the pits were carried down to rock, and on it the timber bents were built.

After the elevated railway columns were underpinned in the first step, the cores of excavation, between the pits and under the columns, were removed, as shown at *A*, and the piece or length of subway or station under each column supported was built. The column was then set on its final foundation on the roof of the subway or station, as shown at *B*. Then the timber bents between *A* and *B* were removed,

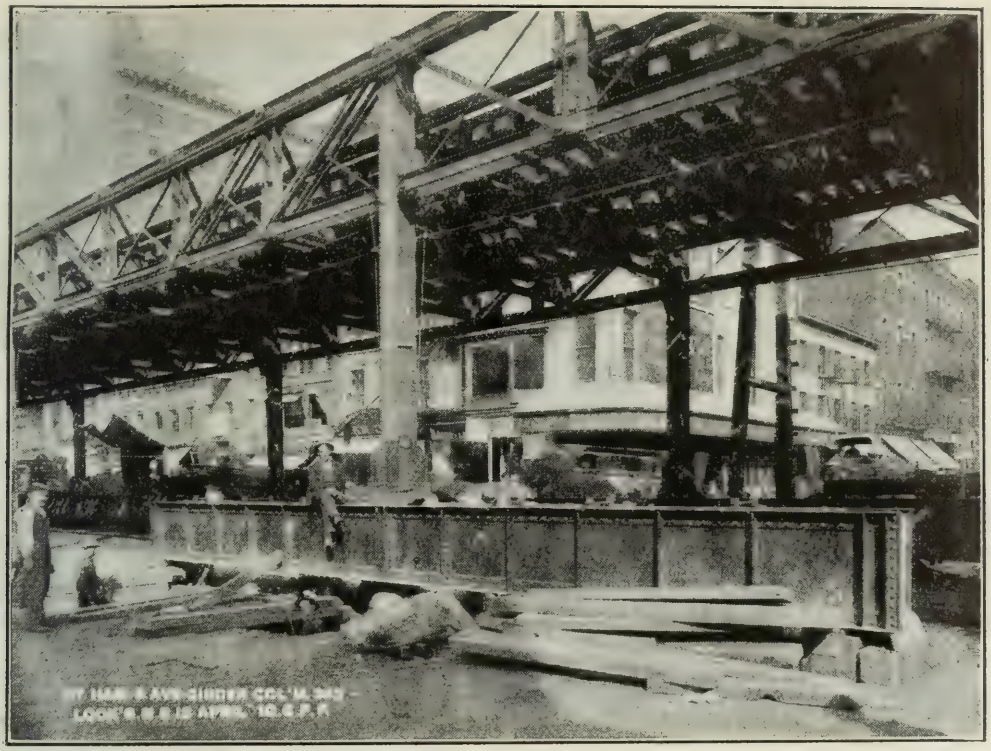


FIG. 5.—COLUMNS OF ELEVATED RAILROAD SUPPORTED ON 36-INCH GIRDERS.

FIG. 6.—ELEVATED RAILROAD SUPPORTED BY OVERHEAD TRUSSES.

and the closure pieces of subway or station between *C* and *D*, were put in, connecting the pieces of subway or station built under the elevated railway columns. The condition after this step was that four of the elevated railway bents, or eight columns, in any one block rested on their final foundations on the roof, and the only remaining columns were those at the street intersections and in the middle of the blocks, which were still on their original brick foundations. The next step, therefore, was to support these remaining two bents or four columns on the other legs of the timber bents already built in the first operation, and remove the cores of excavation under the columns. The pieces of tunnel or station under these columns were then built and the columns were set on their final foundations on the subway roof. The girders and timber bents which last supported these columns were removed and then the closure pieces of subway or station were built. Meanwhile, during the completion of the closure pieces of subway or station, the brick piers for the final foundation of the surface car tracks were built on the roof under each yoke of the surface track structure. The duct benches of the track structure were built on a dry rock packing wall laid up between the brick piers and resting on the roof of the tunnel or station, as shown at *D*.

The sewers were built as the work progressed, and the spaces behind the walls and over the subway or stations were back-filled and puddled to within 7 ft. of the street surface.

Timber bents were built every 6 ft. on each side of the subway on firm foundations of earth or rock, as the case might be, to support the gas and water pipes and electric duct lines, and prevent settlement with the fill.

The gas and water pipes and electric duct lines were then relaid, and the street was back-filled and puddled, up to the surface. A temporary block pavement was laid on the fill at the level of the street surface; this was left in place until the fill had settled sufficiently, and then the asphalt pavement was put down, completing the work.

From 27th to 31st Streets.—The methods and progressive steps in this section were similar to those in the blocks between 12th and 27th Streets, the only essential difference being in the first operation, which was to underpin temporarily the columns of the elevated railway, and, after the removal of the earth excavation, to build

temporary concrete foundations under these columns from the top of the rock, so that they rested thereon and were in safe condition for the excavation of the earth and rock under those adjoining. The foundations of the columns in the center of the cross streets and in the middle of the blocks were carried down to rock temporarily, and then the succeeding operations were similar to those in the work between 12th and 27th Streets.

From 31st to 33d Streets.—The 33d Street Station.—The methods in this section, as shown on Plate VI, differed from those in the work between 12th and 31st Streets, owing to the following conditions: The station occupied the full width of Sixth Avenue, 100 ft.; the station roof was to be of the groined-arch type; and the positions of the elevated railway columns were eccentric with reference to the station columns. For these reasons, it was necessary to build larger pieces of station under the columns in order to provide a safe foundation for them while excavating the adjoining cores. It was found necessary to support the elevated railway columns in such a manner that there would be sufficient space between the temporary column supports in which to build these larger pieces of station under the columns.

The first step was to underpin temporarily the alternate columns and carry their foundations down to rock by temporary foundations of concrete. This, of course, made these columns safe while the intermediate cores of rock were being excavated. Large overhead trusses, resting on the temporary concrete foundations of the alternate columns, were then erected to support the intermediate columns. Then light steel trusses were placed under the street surface planking, spanning the space required for the section of station to be built, and the street surface and all pipes and other properties in the street were supported on them. This left the entire space between the bearings of the light trusses free from encumbrances such as timber or other supports of the street. The rock under the columns supported on trusses was then excavated, and sections of station were built up under them. Then these columns were placed on their final foundations, supported by the steel columns of the station. After this operation, the trusses were removed and the columns, which had been first underpinned temporarily, were again underpinned and the temporary foundations and rock cores were removed from under them. The closing sections

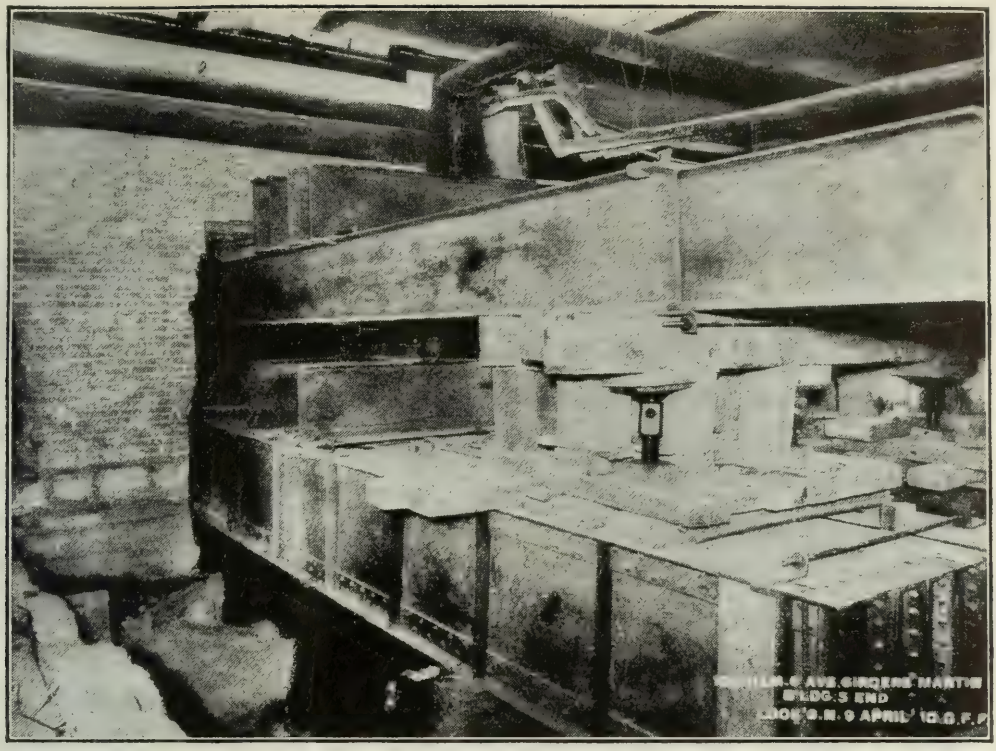


FIG. 7.—STEEL UNDERPINNING OF A COLUMN OF THE MARTIN BUILDING.

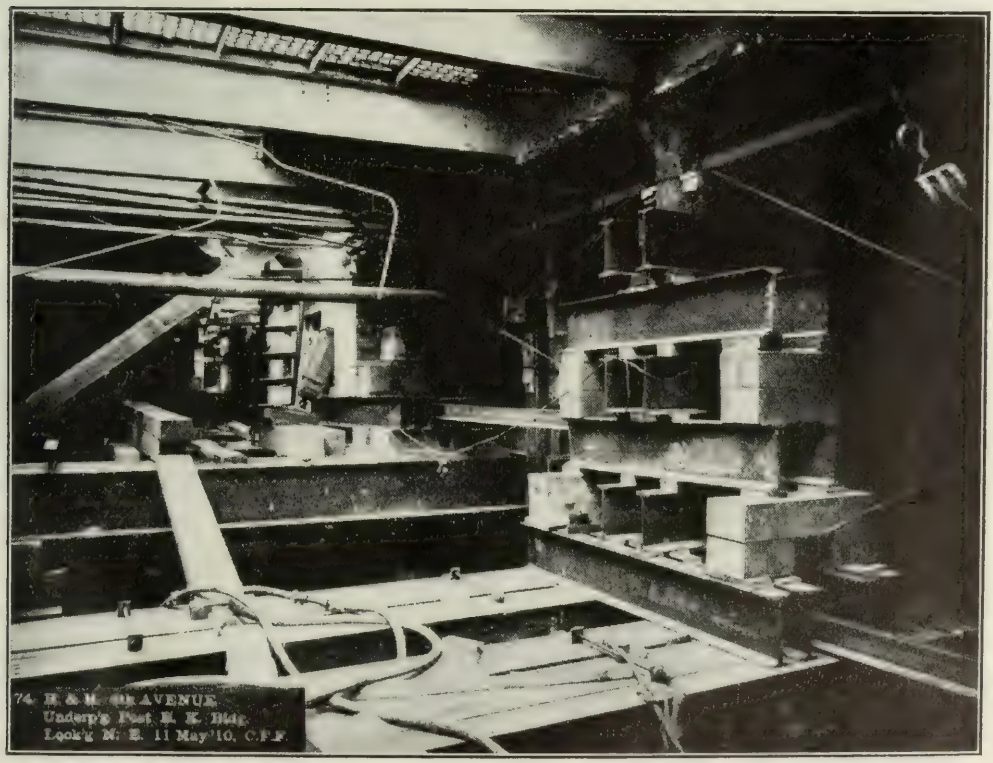


FIG. 8.—STEEL UNDERPINNING OF A COLUMN OF THE BROWNING-KING BUILDING.

or lengths of the station were built under these remaining columns and they were set on their final foundations carried on the steel grillages supported by the steel columns of the tunnel station, thus completing the gaps between the pieces of station first built.

During these operations, the buildings on both sides of Sixth Avenue between 31st and 32d Streets were underpinned. Those on the west side (from four to six stories) were underpinned in the usual manner, and their foundations were carried down to solid rock. The underpinning of the buildings on the east side, was a different matter, as the slope of the rock required that it be removed from under the faces of the buildings and to a distance of from 15 to 20 ft. inside the building line. To insure safety, it was necessary to excavate all the rock to the stratum the toe of which cleared the inside lines of the new foundation walls of the buildings at the tunnel sub-grade.

In underpinning these buildings, the rock excavation for the subway station was carried east as far as the point at which the top of the natural slope of the strata safely cleared the original foundations of the buildings. The columns and walls of the buildings were then underpinned, as shown on Plate VI, and the rock prism was removed so that the natural slope of the rock cleared the line of the proposed wall. Then the foundation was carried down to sub-grade and the station at that place was completed.

The work in the interior of the station was then prosecuted, and the track and other details were completed.

The Crossing of the Pennsylvania Tunnel at 32d Street.—The grade line of the subway passed 5 ft. 6 in. below the top of the roof arch of the Pennsylvania Tunnel. This required that this arch, 5 ft. thick at the crown, be removed and a steel girder construction be built, as shown on Plate II. The elevated railway columns resting on the roof arch of the Pennsylvania Tunnel were supported temporarily and the tunnel arch was removed.

The reinforced concrete arch of the Pennsylvania Tunnel was drilled and blasted away, down to a point 10 ft. above the springing line of the arch, or 2½ ft. above the mezzanine floor, and the girder crossing was built. The columns resting on the girder crossing passed through the station roof and formed the towers on which were supported the foundations of the two elevated railway columns in 32d

Street. The four platforms of the station were supported by these girders and I-beams.

Underpinning the Elevated Railway.

From 12th to 31st Streets.—It will be readily seen that the existence of the Sixth Avenue Elevated Railway structure over the proposed location of the tunnels was the controlling factor in determining the method of prosecuting the work. With the exception of about ten of the lighter columns (those supporting the station platforms beyond the lines of the excavation), the entire elevated railway structure was supported temporarily and placed on new foundations on the roof or walls of the subway.

It was specified by the Interborough Rapid Transit Company that the maximum number of main structure columns removed from their original foundations at any time and supported temporarily should be four bents of two column each, out of the six bents in any block of Sixth Avenue. Between 12th and 31st Streets it was decided to adopt a system of underpinning four bents and leaving two bents on their original foundations for the anchorage of the structure while carrying on the work. In any block, therefore, the usual plan was to leave the columns at the street intersections and in the middle of the blocks on their original foundations and to underpin the other four bents, and, after setting these four bents on their final foundations on the roof of the tunnel or stations, to underpin the first two bents and set the four columns on their final foundations. At some points, such as at 14th, 18th, 23d, and 28th Streets, this system was modified, but, in general, it was followed.

The loading on each elevated railway column was 45 tons on those between the stations and from 45 to 80 tons on those at the stations.

It will be readily seen that, as the elevated railway is a pin-connected structure, it was possible to support it only at the columns.

To support these columns, brackets, with the same spacing of rivet holes as the columns, were made, having from twelve to twenty-four 1-in. holes in each. Rivets were knocked out of the columns of the elevated railway structure at the places where brackets were to be fastened, and the brackets were riveted to the columns through the rivet holes of the columns. The number of rivets in the brackets and the sizes of the brackets depended on the loading of the columns. The

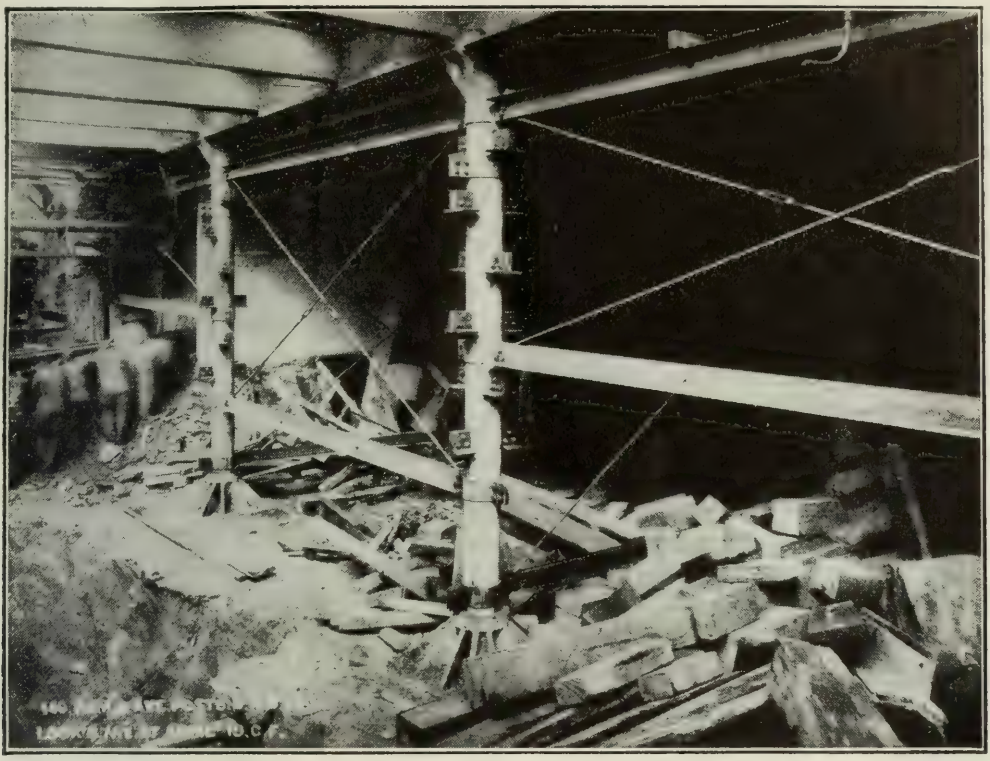


FIG. 9.—STEEL PLUGS AND LATERAL BRACING OF COLUMNS USED IN UNDERPINNING THE BROWNING-KING BUILDING.

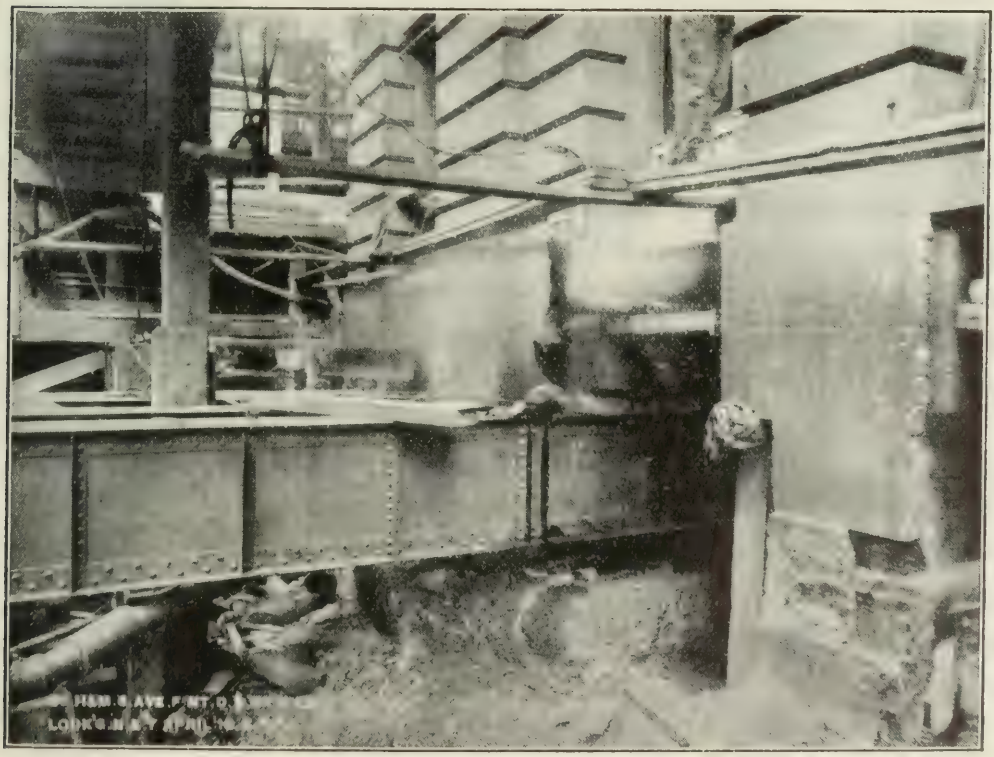


FIG. 10.—GIRDER NEEDLES USED IN SUPPORTING THE BANK BUILDING.

underpinning or framing for the temporary support of each column of the elevated railway was made up of two timber bents supporting the two 24-in. I-beams, 26 ft. long (or 36-in. girders, 39 ft. long), on which rested the column brackets already described. These timber bents were made up of two 12 by 12-in. legs, a 12 by 12-in. cap, and a 12 by 12-in. sill which rested on the concrete floor or on piles in the bottom of the pits dug to receive the underpinning timbers. When wet soil was encountered in the pits at or near the sub-grade of the tunnel, timber piles were driven in the pits down to hardpan or rock, and the timber bents were framed on them. Four timber piles were driven for each timber bent, or eight for each elevated railway column. These piles were driven with an ordinary steam pile-driver running in the leads of a common pile-driver frame provided with drop leads to allow the steam hammer to be lowered below the surface of the street.

Between 17th and 18th Streets, which was one of the localities where the foundation was wet, the station of the elevated railway prevented the use of the pile-driver, because there was not clearance enough for the frame, and here pipe piles were driven with the aid of jacks and a water jet. These pipe piles were driven in the following manner: Two brackets were riveted to the elevated railway columns to be supported, and two 24-in. I-beams were placed with their ends under the brackets. At the other ends of the I-beams, pits were dug to receive the pig-iron counterweights, which were slung to the ends of the beams. A 60-ton hydraulic jack was then set under the beams and on top of the pipe to be driven. The pipe was then jacked down, using the two beams as a counterweight against the resistance of the pile. The columns, if necessary, would have afforded a weight of 60 tons, at one end of the beam, and the weight of pig iron, of course, could be adjusted to meet any weight equal to it. The pipes were driven in small lengths, which were screwed on, one after the other, as each pipe was driven.

As shown on Plates V and VI, the timber bents were braced transversely and longitudinally with 6 by 12-in. timbers, and steel knee-braces were placed between the timber legs and the bottoms of the beams or girders. These braces were made up of two 4 by 3 by $\frac{3}{8}$-in. angles. Between the brackets on the columns and the beams or girders, a 2-in. oak block and two 2 by 24-in. oak wedges were inserted, by which the column was raised in cases of small settlement.

The columns of the elevated railway were maintained at all times within 1½ in. of their original elevations.

At the commencement of the work each column of the elevated railway was supported on two 24-in. 100-lb. I-beams, 26 ft. long, which rested on the timber bents, but this required the digging of two pits between the columns, or two pits for each column. This process was so slow, so expensive in excavation, and reduced so much the sizes of the pieces or lengths of subway to be built under each column in the first operation, that the span of the underpinning beams was increased by substituting for the I-beams two 36-in. girders, 39 ft. long.

It is evident that this improvement in framing the temporary underpinning greatly facilitated the work of building the subway, and effected a large saving in time and money. The digging of one pit 14 ft. wide, north and south, for each column, instead of two pits 8 ft. wide per column, greatly reduced the cost of the excavation of the subway, as it was practically as cheap to dig the 14-ft. pit and erect the timber tower in it as it was to dig one 8-ft. pit and erect the single bent in it with the framing. This was especially noticeable in wet ground.

When the 39-ft. girders were adopted, two timber bents were placed under the ends of the girders, 6 ft. from center to center, and framed together in the form of towers in the pits. This massing of the timber bents and framing removed much of the obstruction to the concrete work of the subway.

At the stations of the elevated railway, the columns had greater loadings. Where there were the heaviest loads, 42-in. girders were used, with the same spans as the 36-in. girders, or the span of the latter was decreased. The 36-in. girders were made up of $\frac{5}{16}$-in. webs, four 6 by 4 by $\frac{9}{16}$-in. base angles, and thirty-two 5 by 3 by $\frac{5}{16}$-in. and $\frac{7}{16}$-in. angle braces or stiffeners. The 42-in. girders were made up of $\frac{3}{8}$-in. webs, four 6 by 4 by $\frac{5}{8}$-in. base angles and thirty-two 3 by 3 by $\frac{5}{16}$-in. and $\frac{7}{16}$-in. angle braces or stiffeners.

From 31st to 33d Streets.—After every alternate column of the elevated railway had been temporarily underpinned, a temporary concrete foundation was built up under it from the rock, so that the columns would be carried safely on the rock while the excavation was being done under the intermediate ones. These temporary concrete

PLATE 10
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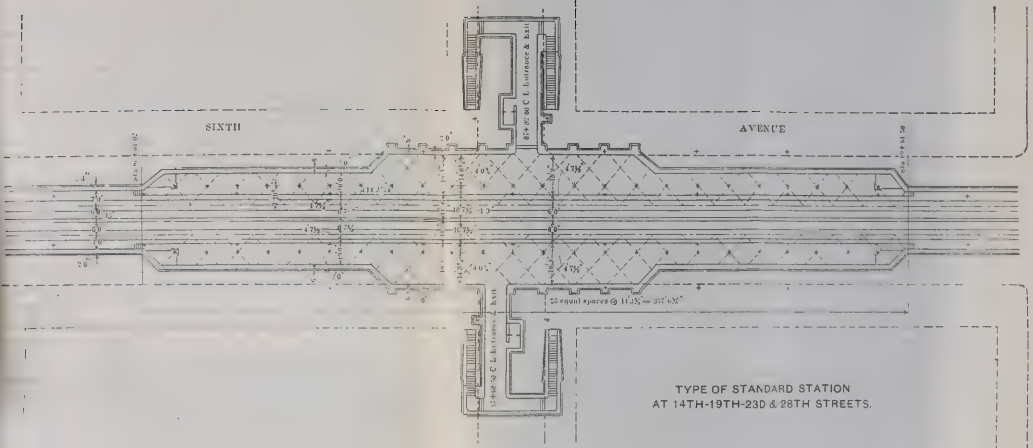
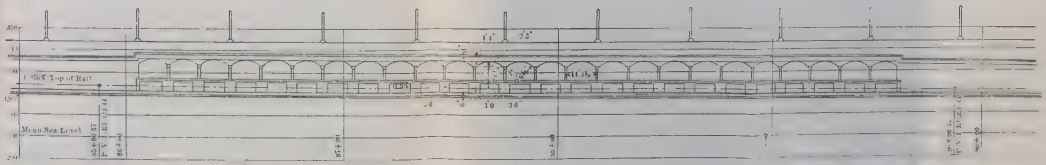
BURROWES ON
SIXTH AVENUE SUBWAY,
H. & M. R. R.





foundations were made large enough to support the ends of the large trusses, as shown on Plate VI. The steel trusses were then erected on the temporary foundations, and had a span of 75½ ft. The top chords of the trusses were bolted to the elevated railway columns, through the rivet holes of the columns (from which the rivets had been removed), and a bracket was riveted to the outside faces of the columns above the outside girders of the trusses. An oak block and oak wedges were inserted between the outside girders and the bracket and between the inside girders and the transverse floor-beams of the elevated railway structure at the columns, to give additional bearing on the trusses. Each truss at each column was made up of two girders, 62 ft. long, supported by two box posts, or legs, framed into the ends of the girders with a bottom chord composed of two 8-in. channels attached to the bottoms of the legs by a pin connection. The pins connected the legs and a cast shoe which rested on a grillage of three 8-in. I-beams, the latter being embedded in the temporary concrete foundations. The girders were made up of ⅜-in. webs, four 6 by 6 by ¾-in. base angles, sixteen 6 by 6 by ¾-in. angle braces, or stiffeners, and six 1-in. cover-plates, three at the top and three at the bottom. The posts or legs were made up of two ¾-in. plates and two 15-in., 35-lb. channels.

On the completion of the lengths of station under the elevated railway columns which were supported by the trusses, the columns were set on their final foundations, supported by the steel columns of the tunnel station, and then the trusses were removed. The elevated railway columns which were on temporary concrete foundations were again underpinned on girders in the same manner as used in the general underpinning between 12th and 31st Streets.

The columns of the elevated railway at the curb lines, which were not main structure columns, were underpinned either in the same manner as between 12th and 31st Streets or were supported on the street surface itself, where it was simply necessary to carry their foundations down to the rock or to the sub-grade of the subway.

Underpinning Buildings.

All the buildings on Sixth Avenue between 31st and 32d Streets, except those on the northeast and northwest corners of 31st Street, which are only 20 ft. front, were supported temporarily while their

foundations were being carried down to rock or to sub-grade. Before the work was started, arrangements were made with the tenants for underpinning and restoring the buildings. On the west side the buildings were of brick, from four to six stories in height. They were underpinned in the usual manner, and before the excavation was well under way in this block, 24-in. I-beams being set under the walls and supported on timber blocking inside the buildings and outside in the excavation for the tunnel. A trench was then excavated to the rock and a 3-ft. concrete wall was built from 2 to 3 ft. below the original foundations, and with its outside face at the building line of Sixth Avenue. The brick walls of the buildings were built on this concrete wall and then the I-beams were removed. The excavation of the rock below the bottom of the concrete wall was done in the usual manner by "line-holing" the rock to make a straight and even break, as the station extended the full width of Sixth Avenue and to within 7 in. of each building. The reason for carrying the foundations of the buildings on the west side of Sixth Avenue to the rock only, and not to sub-grade, was the fact that the strata dipped northeast, and thus aided in making the excavation of rock in front of the building safe.

The four buildings underpinned on the east side were: A 20-ft. front, four-story brick, between two large buildings; a 59-ft. front, eleven-story, steel frame building, designated as the "Martin Building," which was shored at all points, with the exception of the south wall; a 59-ft. front, eight-story, steel frame building, with cast-iron columns, designated as the "Browning-King Building"; and a 40-ft. front, five-story, granite and marble-faced building, without steel framing, designated as the "Bank Building." These buildings are shown on Plate VI.

The rock was removed as close to the existing foundations as safety allowed, the buildings were shored up, the rock was removed from under their fronts, and then their foundations were carried down to sub-grade.

The Martin Building.—The front of this building was carried on four framed steel columns, the two end ones being embedded in the side-walls. It was necessary to shore up the north wall and column and the two center columns. The south wall and column rested on rock, and, being at a safe distance from the excavation, required no

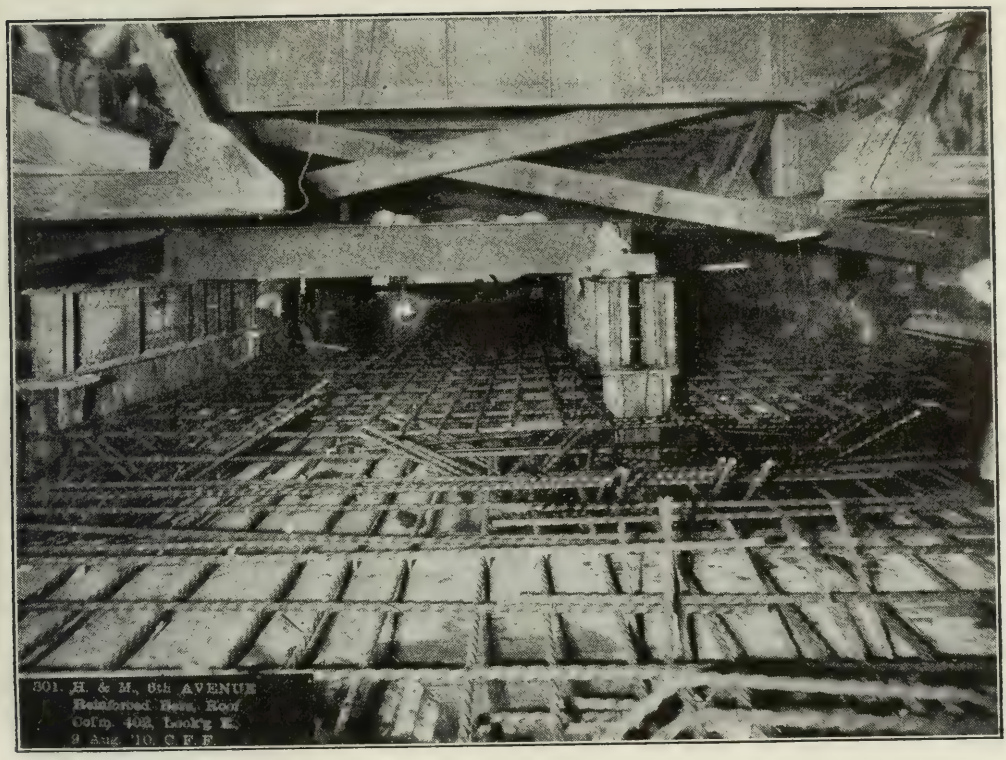


FIG. 11.—REINFORCING RODS, GROINED ARCH FORMS, AND STEEL GRILLAGES FOR FOUNDATIONS OF ELEVATED RAILROAD COLUMNS.

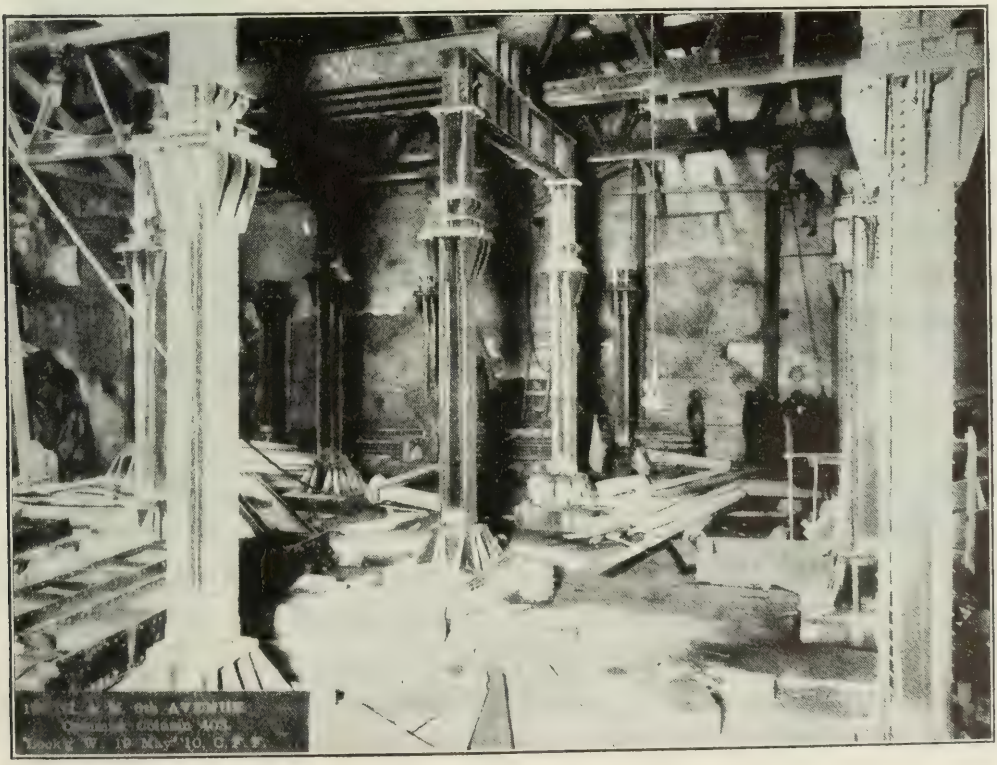


FIG. 12.—COLUMNS, WITH GRILLAGES, FOR SUPPORTING COLUMNS OF ELEVATED RAILROAD.

support. The three columns shored up were of a steel box, riveted type, 16 by 14 in. Rivets were removed from each column, and four steel brackets, aggregating 106 $\frac{3}{4}$-in. rivets per column, were riveted to them. Seven 36-in. girders, 39 ft. long, similar to those used in underpinning the elevated railway between 12th and 31st Streets, were placed on each side of each column. These girders, fourteen under each column, were supported at the outer ends by a pair of heavy 12 by 12-in. timber bents, parallel to the building front, in the excavation already made and 20 ft. from the building columns. At the inner ends they were supported by a reinforced concrete beam or girder, built on the rock, as close as possible to the second line of building columns and parallel to the building front. These 39-ft. girders had a span of 34 ft. between the center of the bearings on the concrete girder and the center of the timber bent. Across each of the fourteen girder clusters there were laid four 24-in., 100-lb. I-beams, two on each side of the column, on which rested the lower two brackets fastened to the building column. Above these I-beams, and parallel to the girders, there were twelve 24-in., 100-lb. I-beams, six on each side of each column, which were supported by the girders through timber blocking. These beams also supported the front wall of the basement. Across these I-beams there were laid four 24-in., 100-lb. I-beams, on which rested the upper two brackets of the building column. An additional support of the columns was afforded by using twelve suspender bolts (four 2-in. round and eight $1\frac{1}{2}$-in. round), which held up the five 15-in. I-beams of the original grillage under the base casting of each building column. The top nuts of the bolts took bearing on the I-beams of the steel above. The proper distribution of the load over the parts of the steel underpinning was secured with wedges and the suspender bolts supporting the bases of the columns. The building columns were kept at their proper elevations with jacks and blocking under the ends of the girders.

After the building columns were picked up, the old foundations and the rock were removed, and, in this building, the columns were extended down to sub-grade with steel columns made up of three 24-in., 100-lb. I-beams, riveted together in the form of an I, and latticed and braced, as shown on Plate VI.

After the excavation was completed, new grillages, made up of five 20-in., 80-lb. I-beams, 10 ft. long, were built under each column

at sub-grade and embedded in concrete; the extension columns were set in place, anchored to the new grillages, and bolted to the grillages of the old foundations. Then a concrete wall was laid with its outside face at the building line, filling the entire space of the excavation over the slope of rock. This wall extended up to the old foundations, which were then grouted on it, and, after the concrete had set, the shoring was removed and the subway station work at this point was prosecuted to completion.

The load on each of the columns of this building was calculated to be 400 tons. Of course, the loading on the wall columns was partly taken up by the brickwork of the north wall of the building, part of which was supported on the steel underpinning and part remained on the original foundation resting on rock.

The Browning-King Building.—The entire front of this building was supported by four cast-iron columns, 11 in. in diameter and 2 in. thick, two in the center and two embedded in the side-walls. The method of shoring this building was similar to that used in the Martin Building (concrete girder and timber bents), except that the details of the steel were different and the columns were not extended to sub-grade.

A similar concrete wall, filling the entire space of the excavation back of the building line, was built, the columns were grouted on top of this wall, and then the steel underpinning was removed.

The method of picking up the columns was as follows: As shown on Plate VI, six holes were drilled completely through each cast-iron column, and then enlarged so as to pass 6 by 1¼-in. steel plugs. These plugs were 24 in. long, and were placed in the holes on edge, 18 in. apart, vertically.

At each end of each steel plug, two 6 by 4 by ¾-in. angles were bolted to give greater bearing on the I-beams supporting them. There was one 15-in., 60-lb. I-beam on each side of each column, supporting the steel plugs at each end, arranged in the form of towers. An equal bearing was maintained on these beams with blocking and steel wedges, so that the steel towers might be considered as one mass fastened to each column. The load was transferred through the 15-in. I-beams and blocking down to 36-in. girders similar to those used in the Martin Building. Ten girders, five on each side of the column, supported each of the two center columns; five girders and seven

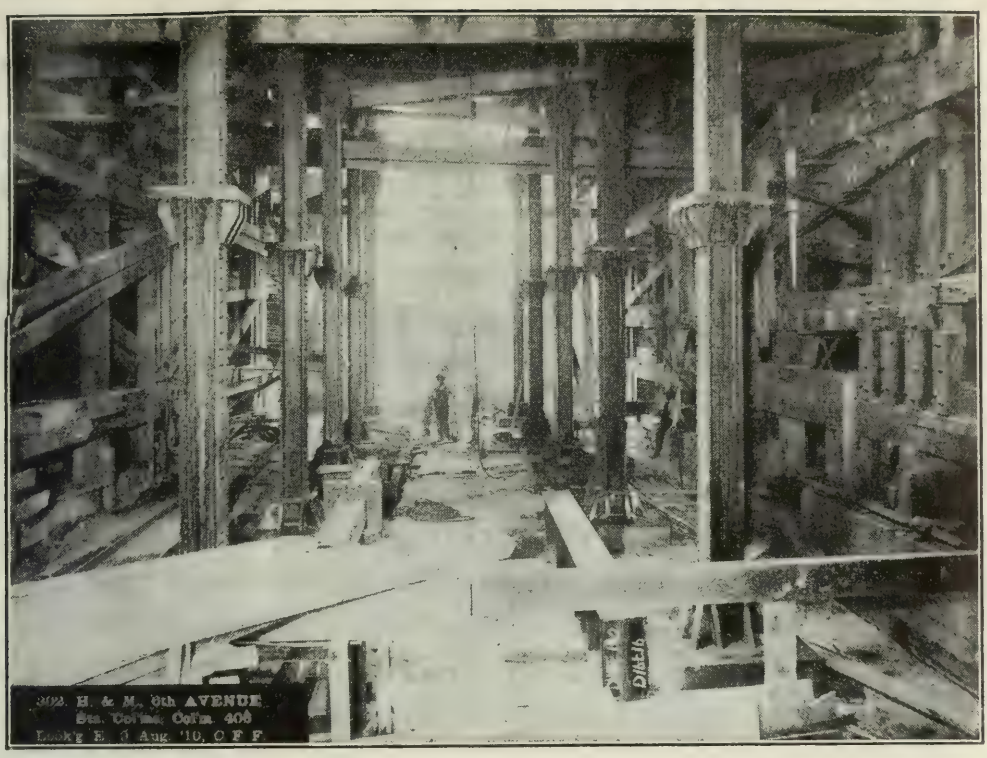


FIG. 13.—CLOSURE SECTION OF THIRTY-THIRD STREET STATION. STEEL COLUMNS SET UP.

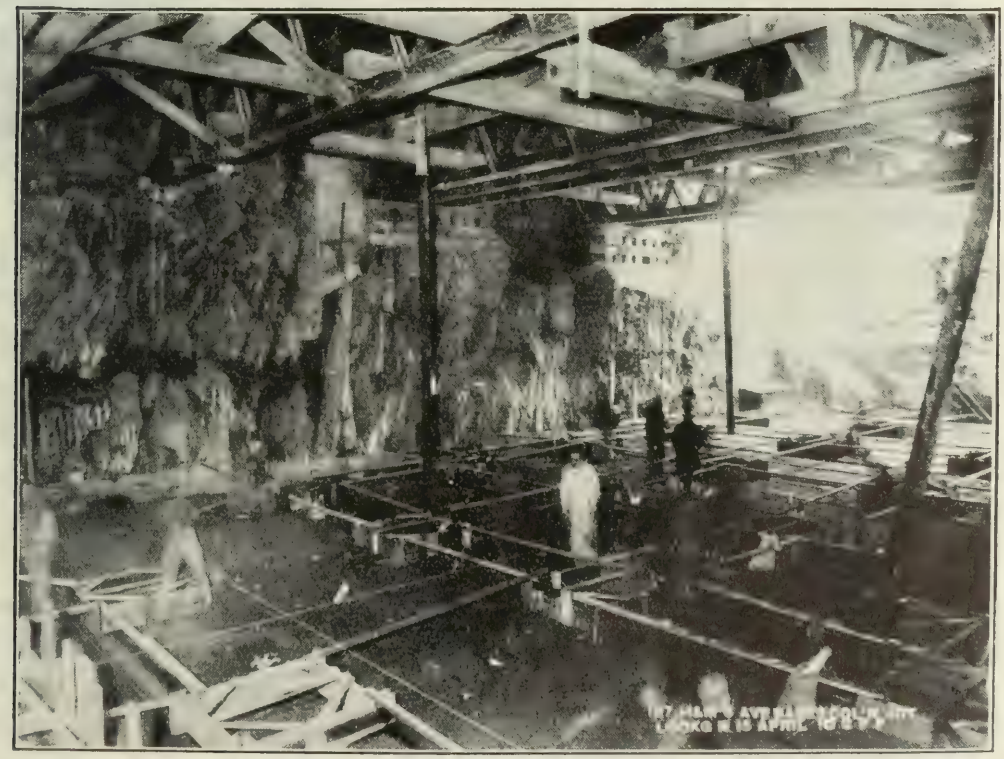


FIG. 14.—CONCRETE FLOOR AND ROCK CORE BETWEEN SECTIONS OF THIRTY-THIRD STREET STATION.

24-in., 100-lb. I-beams supported the south wall and columns of this building and the north wall of the four-story building; and four girders and eight 24-in., 100-lb. I-beams supported the north wall and columns of this building and the south wall of the Bank Building. These girders had a span of 34 ft. between their bearings on the concrete girder and the timber bents. Additional support was obtained with four 2-in. round suspender bolts supporting six 9-in., 30-lb. I-beams placed under the base castings of the building columns, taking bearing at the bolt heads on the lower set of I-beams of the steel tower. These 9-in. grillages under the column bases were placed after the columns had been picked up on the steel towers, and were simply an additional precaution. After the removal of the shoring, the steel plugs were sawed off at the faces of the columns. The building columns were kept at proper elevation with jacks placed under the ends of the girders, as in the Martin Building.

The load on each of the columns of this building was calculated as between 240 and 260 tons.

The Bank Building.—As this building had no steel framing, a different method of support was necessary. The underpinning cared for the entire front of the building and a short length east of the corner. The foundation was composed of four piers or columns of granite-faced brick masonry. The front was carefully tied together with wire rope fitted with turnbuckles.

Owing to the condition of the excavation and the order of building the different pieces of station, it was found that the excavation in front of the bank could not be completed soon enough to allow sufficient time for the underpinning of the building and the subsequent construction of the piece of station opposite it. If the underpinning had been left until the excavation had been made in front of the building, the completion of the station and the opening of the tunnel for traffic would have been delayed. Owing to these conditions, two reinforced concrete girders were built on top of the rock, and the steel girders used in underpinning the building were supported on them. One of the girders was built inside the building and the other outside, and both were parallel to the building front. A concrete wall was then built in the trench excavated under the building front, up to within 2 ft. of the bottom of the steel girders supporting the build-

ing, and the foundations were carried down with brick masonry to the top of the wall, with the gradual removal of the girders.

At the outside edges of the piers of the foundation, recesses or steps were cut deep enough to allow the placing of one girder on each side of each pier. The remainder of each pier was then gradually removed, substituting the girders for the brickwork, until the piers were supported entirely on the girders. Eighteen steel girders were used in supporting this building, with the aid of the eight 24-in. I-beams and four girders used for the north wall of the Browning-King Building and the south wall of the Bank Building. The rock trench was line-holed along the inside face of the building and the face of the outside concrete girder, in order to provide an even break near the concrete girders, and the rock faces of the trench were braced with timber to prevent the rock from slipping.

Owing to the removal of the steam-heating plant of this building, which was originally under the sidewalk, it was necessary to erect a 50-h.p. boiler on the street to supply heat during the winter and until a new heating plant had been installed in the basement.

The four-story brick building between the Browning-King and Martin Buildings was supported on the underpinning used for the walls of the adjoining buildings.

A factor of safety of four was used in calculating the strength of the steel underpinning systems. The buildings were maintained to within $\frac{1}{4}$ in. of their original elevations by the jacks. The girders and I-beams were used several times, as the buildings were not all underpinned at the same time. There were no cracks or other damages due to settlement, and the interiors of the buildings were restored to the satisfaction of the owners.

Excavation.

The excavation was started in a series of pits to contain the elevated railway supports. These were usually 8 ft. wide, extended across the street the full width of the work, and were carried down to sub-grade in cases of dry soil, and to within from 8 to 12 ft. of sub-grade in cases of wet soil, where piles were required to support the elevated railway.

The excavation of the cores of earth or rock under each pair of bents of the elevated railway were generally from 100 to 110 ft. long,

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PLATE IV.
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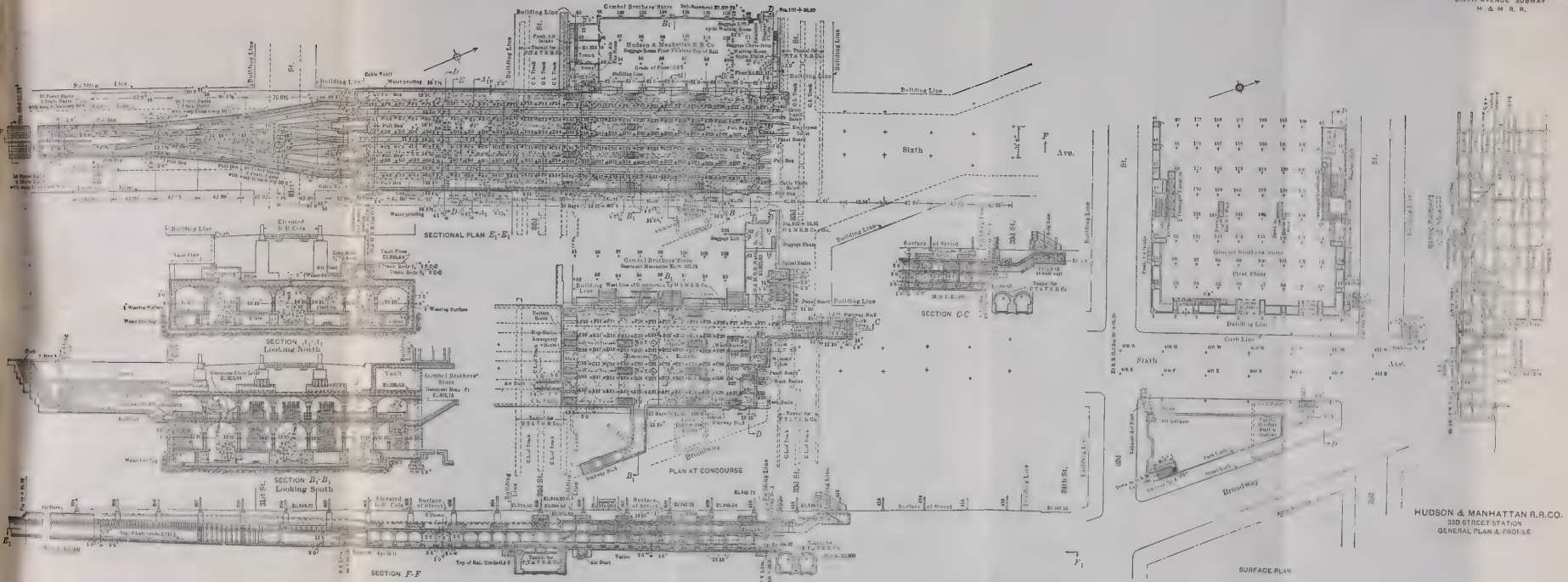
BURROWES ON
SIXTH AVENUE SUBWAY
H. & M. R. R.



HUDSON & MANHATTAN R.R.CO.
33D STREET STATION
GENERAL PLAN & PROFILE

from north to south, depending on the depth, and were of the full width required for building the subway and including the sewers. When these large pits were finished to sub-grade, the lengths of subway were built in them, and the elevated railway columns were set in their final position on the roof. The intermediate cores of excavation between these large pits were then removed to sub-grade, thus completing the excavation.

The timbering is shown on Plates V and VI. There were two or three lifts in the sheeting of the cuts, depending on the depth of the excavation and the elevation of the sewers alongside the subways. Where the soil was wet the lower lift was made up of 3-in. tongued and grooved, yellow pine sheeting. The sheeting for dry soil was of 2-in. yellow pine.

The bracing of the cut, as shown on Plate V, was designed so as to interfere as little as possible with the laying of the subway concrete, but the braces were left in place and allowed to pass through the concrete roof where there was any advantage in the saving of time or cost. The bents or timber sections were made up of 10 by 10-in. rangers and 12 by 12-in. braces and posts, and were generally placed 10 ft. apart. The details of the timbering are shown on Plate V, and need no special description.

The temporary planking of the street surface was laid on 10 by 10-in. and 12 by 12-in. stringers or deck pieces running longitudinally with the work, and on them was placed 4- and 5-in. planking laid transversely across the stringers. This deck or planking was supported by the posts of the cut bracing. As this planking was necessarily in service a long time, and subjected to the wear of the street traffic, it was necessary to watch it constantly and renew it in order to avoid accident.

When the 39-ft. girders were used in underpinning the elevated railway columns, as shown on Plate V, the timbering of the cuts was greatly simplified during the rock excavation between stations by using six 24-in., 80-lb. I-beams, 35 ft. long, in three pairs, each pair supported by one 12 by 12-in. post under the ends, resting on the rock. These I-beams supported the street surface and permitted the elimination of the posts necessary to support it. The bracing of the sheeting was carried across the cuts as usual, but required very few posts.

The materials through which the excavation was made were generally dry sand and loam, but between 12th and 13th Streets a fine sand with water was encountered at sub-grade, below tide level, and it was necessary to lay a 10-in. timber floor in the bottom of the cut on which was laid a 3-in. planking to furnish a proper foundation on which to build the tunnel. It was also necessary to provide temporary drains to carry off the water while laying the concrete in this part. Between 15th and 18th Streets a wet sand was encountered at sub-grade, but in this case it was simply necessary to put in a floor of 3-in. planking on which to lay the concrete floor. A small outcrop of rock was removed at sub-grade at 14th Street, but the rock excavation really commenced at 20th Street and continued north throughout the work to 33d Street. The rock varied in depth from 6 ft. above sub-grade at 20th Street to 35 ft. above sub-grade at 33d Street, and was very uneven, at times running just below sub-grade, and requiring a deeper excavation at these points.

Rock Excavation.—The rock formation was the usual mica schist, the general slope of the strata being about N. 30° W.; the inclination was between 15° and 20° east of vertical.

The bench rock was drilled with 2½- and 3-in. drills, and the boulders and large pieces with "Little Wonder" drills. It was blasted and broken into small pieces suitable for dumping into the trucks. The drill holes in the bench rock were generally 14 ft. deep. Where the temporary foundations of the elevated railway columns were on the rock, the excavation between these columns was carried as close to them as safety allowed, and lines of holes were drilled and blasted transversely across the cuts so as to make an even break and give as much space as possible in the excavated pits for building the pieces of tunnel or station. The rock excavation was carried toward the east on account of the slope of the strata.

From 31st to 33d Streets.—The 33d Street Station.—In this section the methods of excavation were similar to those described for the work between 12th and 31st Streets, excepting that, on account of the larger pieces or lengths of station required to be built in the first operation, as described in "Underpinning the Elevated Railway," the method of underpinning the elevated railway made the pits of shorter length from north to south.

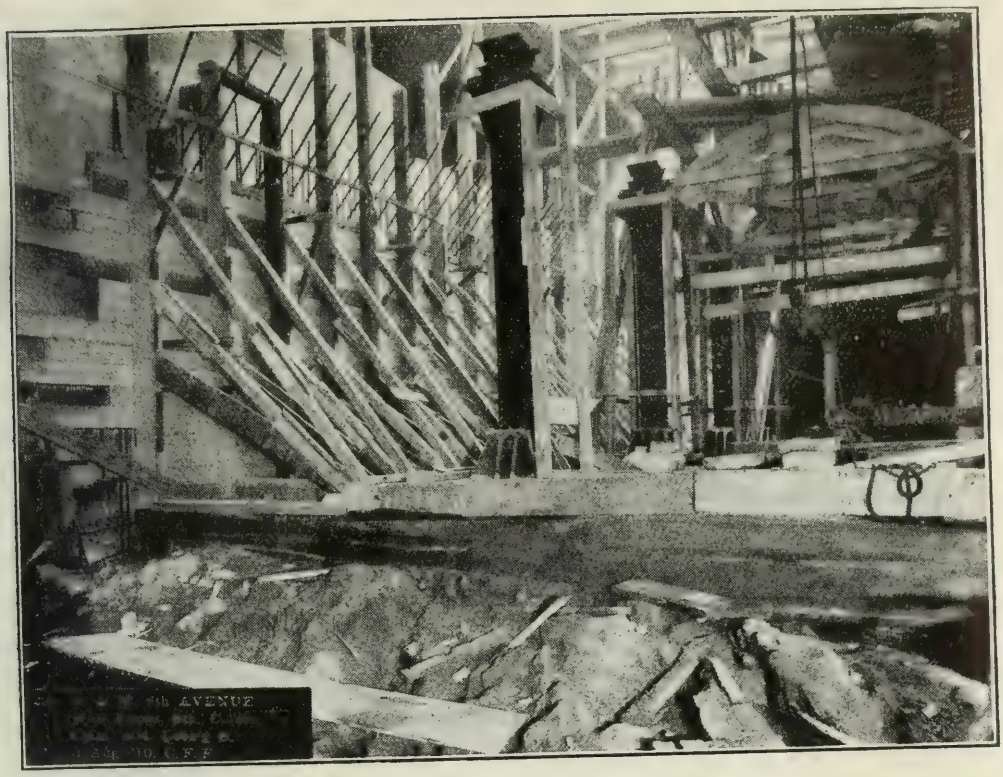


FIG. 15.—WALL AND ROOF FORMS, THIRTY-THIRD STREET STATION.

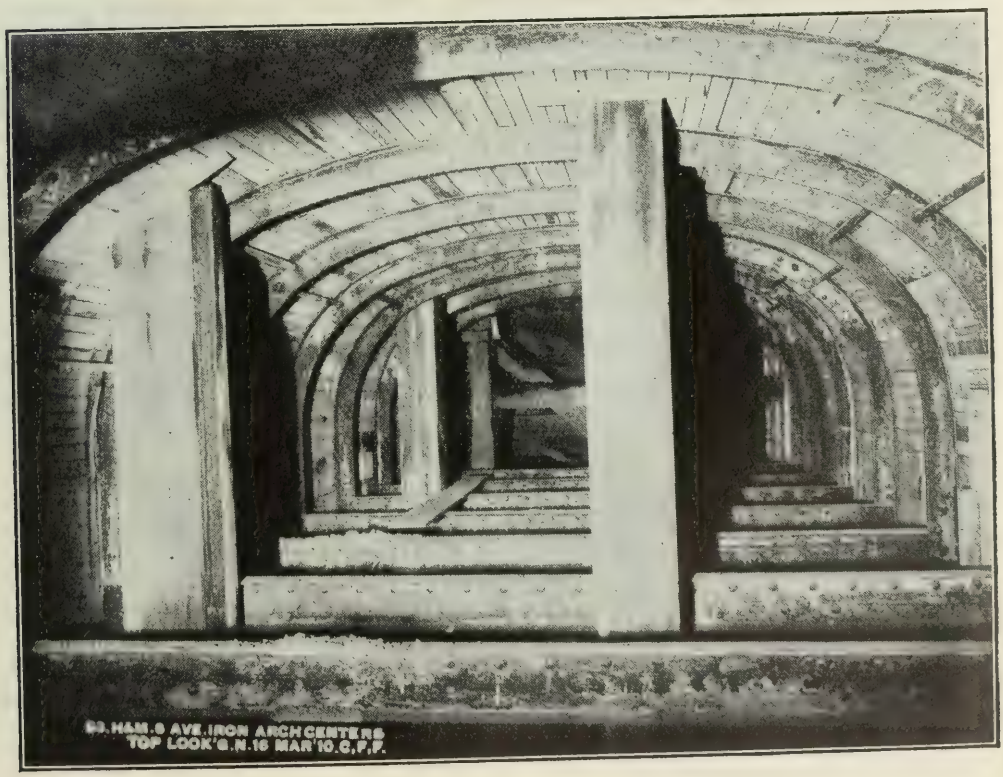


FIG. 16.—STEEL RIBS AND LAGGING OF FORMS IN STANDARD SUBWAY SECTION.

As shown on Plate V, the timbering and support of the street between 31st and 33d Streets differed from that of the work from 12th to 31st Streets. Light steel lattice trusses, 8 ft. deep, were erected under the street planking, and the surface car tracks, pipes, electric properties, and the street surface planking were all supported on these trusses. This eliminated all the obstructions caused by the usual method of bracing the cuts with timber, and furnished a clear space in which to build the subway or pieces of station under the columns. As shown on Plate VI, each lattice truss was made up of a bottom chord, two 8 by 8 by $\frac{11}{16}$-in. angles and one 7 by $\frac{9}{16}$-in. web-plate between the $\frac{9}{16}$-in. gusset-plates connecting the diagonals to the chord. The top chord consisted of two 8 by 8 by $\frac{1}{2}$-in. angles and one 16 by $\frac{9}{16}$-in. web-plate. The bracing or diagonals were made up of two angles, 6 by 4-in. and 4 by 3-in., from $\frac{3}{8}$ to $\frac{3}{4}$ in. thick. These trusses were made up in small pieces in order to facilitate their erection under the street planking. When the lengths of subway or sections of station had been built under these trusses, and the elevated railway columns had been set on their final foundations, the posts for the street pipes, street surface, etc., were carried down to the roof of the completed work. The trusses were then removed. After the columns, resting on temporary concrete foundations, were again underpinned, the foundations and cores of rock under them were removed.

Construction.

Concrete.—The concrete used on the work was mixed on the street surface in Ransome, $\frac{3}{4}$-yd. mixers. The proportions of concrete for foundations, walls, and roofs was 1 to 6, varying from 1 cement, 2 sand, and 4 broken stone to 1 cement, $2\frac{1}{2}$ sand, and $3\frac{1}{2}$ broken stone, according to the voids in the sand and stone. The concrete used for track floors on bed-rock, and for other floors or places where there was little strain or load, was composed of 1 cement, 3 sand, and 5 stone.

The stone was $\frac{3}{4}$ and 1-in. and run-of-crusher Hudson River limestone. It was found that the $\frac{3}{4}$-in. stone first used contained such a large percentage of screenings and dust that it was advisable to get 1-in. stone, ranging from $1\frac{1}{4}$ to $\frac{1}{2}$ in., with a small percentage of screenings and a very small percentage of dust. This stone was adopted, for the concrete work throughout, as being better for a

1 to 6 mixture, under the conditions of the work. The stone and sand were delivered alongside the dock and stored at the yards. The stone used in the street restoration, being required in small quantities, was delivered directly on the work.

The sand was obtained mostly from Hempstead Harbor, Long Island; it was of a good coarse quality and contained from 2 to 3% of loam. The maximum quantity of loam allowed was 2%, and, during the first two years of the work, it was very hard to obtain sand of this quality, but there was no difficulty during the last three years.

"Giant" cement was used throughout the work. It was tested at the company's laboratory in Jersey City, and delivered at the yard in trucks.

The cement was tested according to the Am. Soc. C. E. standard. It was uniform in quality, and there was no difficulty in obtaining a reliable set or proper age.

The water for the concrete was obtained from the city mains through metered connections at convenient points along the work. The mixers were run with compressed air obtained from the contractor's power plant at the 42d Street yard. The mixing gangs consisted of from 14 to 18 men, as follows:

One mechanic running the mixer,
One skilled laborer dumping the cement and handling the water,
From 6 to 8 men loading wheel-barrows, and feeding sand and stone to the mixer,
From 6 to 8 laborers delivering concrete from the mixer at the point of laying.

The sand and stone were measured and delivered to the mixer in wheel-barrows. The concrete was carried from the mixers to the points of laying in "Ransome" concrete carts, and dumped into chutes at the street surface.

The sizes of the gangs placing the concrete varied considerably with the number of mixers supplying concrete and with the size of the piece laid, the difficulties in placing, and the length of haul between the mixer and point of laying. All the concrete was mixed "wet" and of such consistency that it would flow readily through the chutes and be handled easily. The length of haul of the concrete

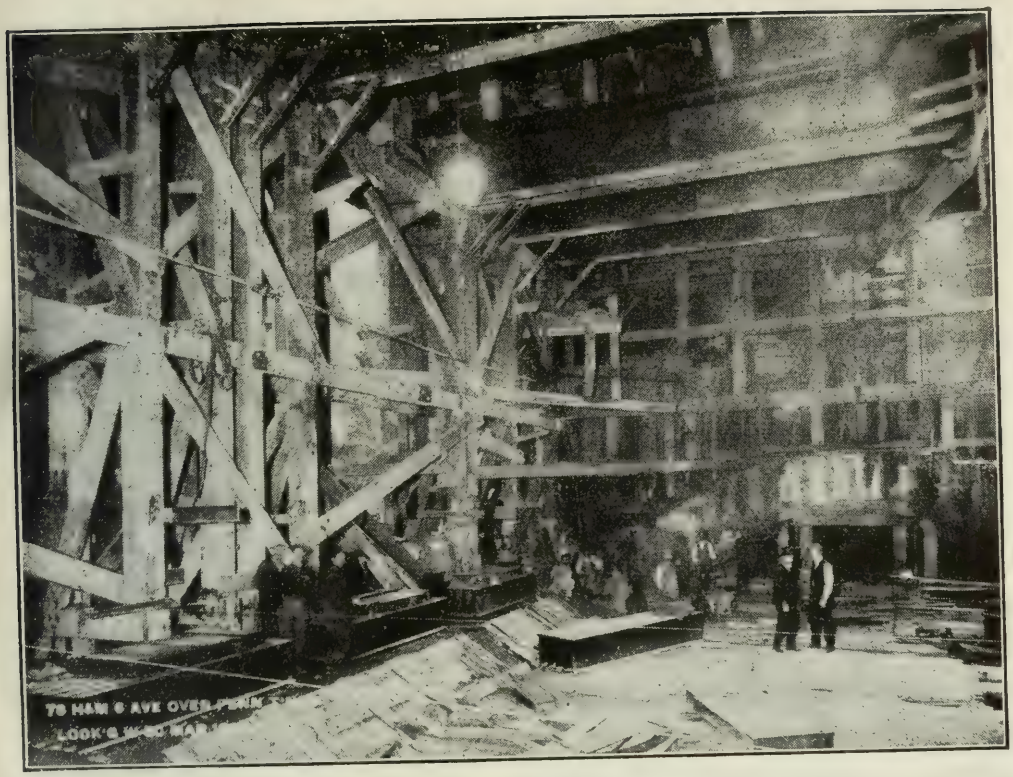


FIG. 17.—PLACING STEEL GRILLAGES FOR SUPPORT OF GIRDERS OVER PENNSYLVANIA RAILROAD TUNNEL.

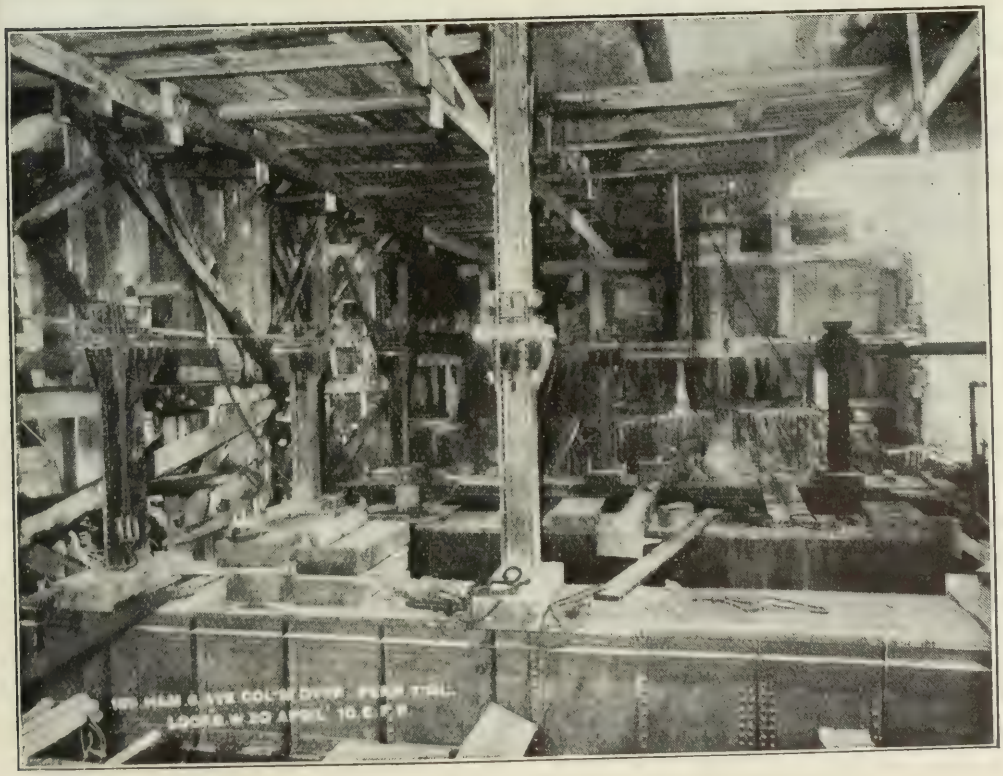


FIG. 18.—STEEL CROSSING OF PENNSYLVANIA RAILROAD TUNNEL.

was from one to two blocks, or from 260 to 520 ft. There was little delay in delivering the concrete from the mixer to the points of laying, because the men could wheel the carts over almost any portion of the street surface.

Owing to the great number of rods in the reinforcement, it was difficult to handle the concrete in the forms for the roofs and walls, and great care had to be exercised not to displace the rods or crowd them against the forms.

All the concrete of the subway was laid under the street planking, and great care was necessary to prevent dirt or chips from being dropped into the forms. No face mixture was used; the concrete was simply rammed or "quaked" by the use of long 2 by 4-in. timbers in the wall work and rammers in the floor and roof work. The faces of the walls and roofs showed very few voids or dry spots after the removal of the forms. The form centers in the standard tunnel section were composed of steel ribs made up of two 5 by 3-in. steel angles. These ribs extended from the toes of the center- and side-walls up to and including the arches. The inside face planking of the walls and the arch lagging were fastened to them with wire staples. The inside wall face forms were of 2 by 10-in., tongued and grooved spruce, and the arch lagging of beveled spruce 3 in. thick.

In building the groined arch forms of the station roofs, the ribs were placed under the groin lines of the arches and were of 2-in. spruce supported by 4 by 4-in. spruce posts. The lagging was beveled, planed spruce, with butt joints, and was nailed to the spruce ribs. The face planking and lagging were used over again as often as their condition allowed, but no attempt was made to redress the forms, as there was little economy in it, there being plenty of places where the rough forms were good enough in the exposed faces of the work.

It is a feature of this concrete work that the forms failed or slipped only four times during the entire work.

The concrete was under the most favorable conditions for setting, as it was under the street planking, thus avoiding sun drying and quick setting and being aided by the drafts of cool moist air circulating through the cuts and finished pieces of tunnel.

The concrete of the subway and stations was laid up in three lifts: first, the floors and foundations; second, the walls from the founda-

tions to 1 ft. below the springing lines of the arches; and third, the roofs.

Concrete was not laid when the temperature was lower than 25° Fahr., and in all cold weather the concrete was covered with salt hay to prevent freezing. Salt was not used in mixing the concrete. The temperature in the pits was very seldom lower than 30° Fahr.

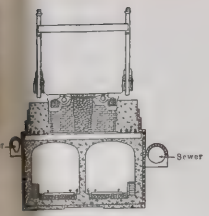
The forms were dressed with soap or paraffin oil in all the work, excepting the arches of the 28th and 33d Street Stations, where the dressing with oil or soap was abandoned in order to obtain a better bond between the white cement finish and the concrete in whitewashing the interiors of the stations.

The steel and cast-iron columns of the stations were filled with 1:3 mortar grout poured in from the tops; this was done in two operations in order to permit the first batch to settle.

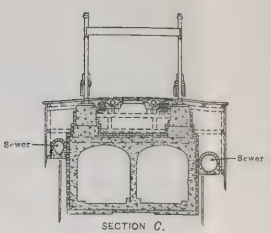
The spaces between the girders spanning the Pennsylvania Tunnel at 32d Street were grouted with neat cement. Mortar could not be used in this grouting because the spaces between the angle stiffeners were so small that mortar was too thick to form a certain enough grout. In this work the cement grout was poured into a 2-in. pipe at the street surface, which gave a head of 35 ft. for the pressure required to fill completely the spaces between the girders.

Reinforcing Rods and Expanded Metal.—The reinforcing rods were 1-in. and ¾-in. square Ransome twisted steel for the walls, roof, and foundations, and ⅜ and ½ in. for the platforms and thin walls. All the rods were delivered alongside the company's dock in lengths ranging from 10 to 30 ft. Rods were bent to special shapes in the yard, by using a bender made on the work and having a hydraulic jack. The rods requiring a bend up to about 45° from the straight were bent by hand. All the lengths and shapes represented certain positions in the walls, roofs, etc. The plans showed continuous rods around the shell of the subways and stations, and longitudinally, so that it was necessary to break the rods at points where it would do the least harm, and bond the breaks by an overlap of 40 diameters of the rods. The rods of the walls, roof, etc., were held 2 in. from the forms by using small mortar blocks laid between them and the forms. The rods in the roof were supported by the forms and by wire hangers fastened to timbers above. The weight of rods per cubic yard of concrete in the station roofs was very high, and the weight of the

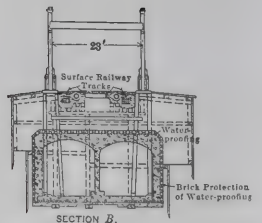
PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No 1239.
BURROWES ON
SIXTH AVENUE SUBWAY,
H. & M. R. R.



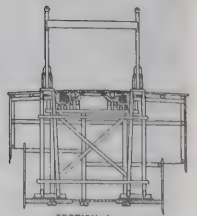
SECTION *D*.



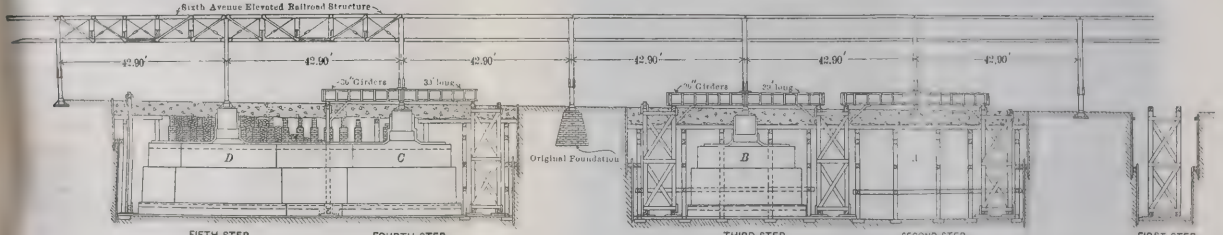
SECTION *C*.



SECTION *B*.

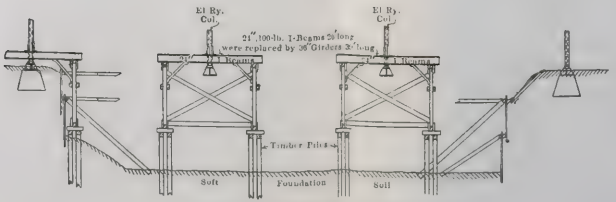


SECTION *A*.

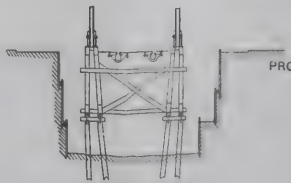


FIFTH STEP. FOURTH STEP. THIRD STEP. SECOND STEP. FIRST STEP.

Timber Bents for Underpinning Elevated Railroad

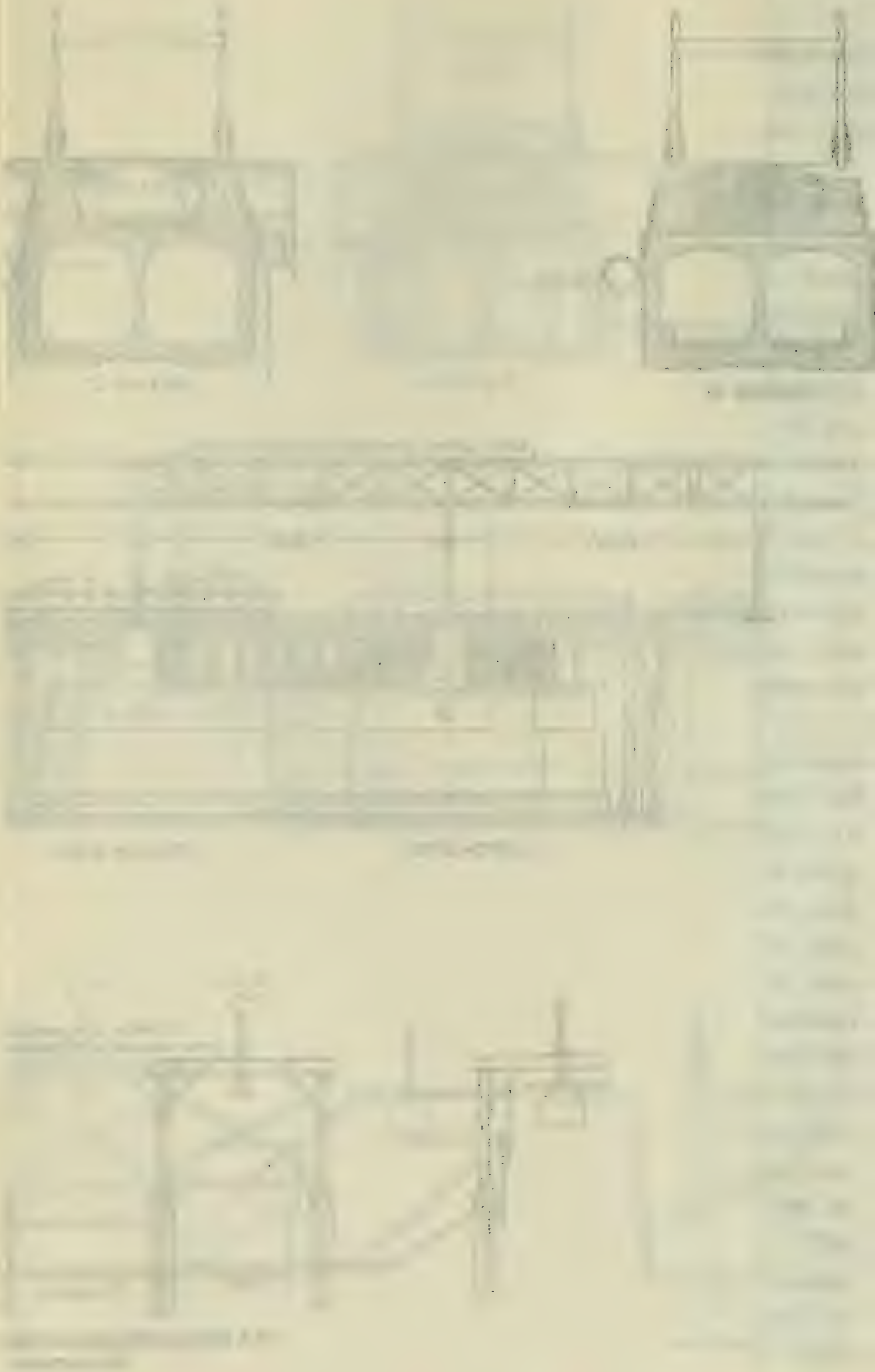


PILE BENTS FOR UNDERPINNING ELEVATED RAILROAD WHEN IN SOFT GROUND.



SECTION OF PILE BENTS.

PROGRESSIVE STEPS & METHODS
12TH TO 31ST STREETS.



concrete and rods combined required that the forms of the roofs be well supported by the 4-in. posts. The average spaces between the rods through which the concrete was poured was 12 by 12 in., and particular attention was given to spacing the rods properly so that the concrete could be placed in the most economical way. The clusters of rods (in the form of baskets) over the station columns were especially difficult to place and keep in position, because the concrete dumped into the haunches of the arches tended to shift the entire cluster so that the rods were displaced or forced against the forms.

Expanded metal or wire cloth was laid under the roof beams of all the flat arches to bond the concrete cover of the beams to the concrete between them.

Brickwork.—The bricks used in underpinning buildings, in the foundations of the surface railway, and in the stations, were the usual standard house brick, and were laid in 1:3 mortar. The partition walls of the stations, booths, ticket offices, etc., were built of 3-in. and 4-in. hollow tile.

Water-Proofing.—The entire work between 12th and 27th Streets was water-proofed with "Siastex," a burlap and asphaltum process. A strong and durable burlap, of more practical value than the usual felt, was used because of the necessity of handling the reinforcement rods over it on the floors and walls. The normal thickness of the water-proofing was 2-ply with felt "plus ply." This was increased to 3- and 4-ply where water pressure and flow were met. A single ply of felt "plus ply" was first laid under all the water-proofing where the concrete surfaces were wet, and over this were laid the burlap and tar. On the roof of the 23d Street Station and under the floor of the enlargement at 12th Street, where there was a flow of water and very wet conditions, a brick and mastic was used. This consisted of a tarred felt "plus ply," on which was laid hot bricks which were grouted with pitch. In all the water-proofing in which the burlap was used, an armor course of 3-in. hollow tile or 4-in. brick was laid to protect it. In the floors, a 6-in. concrete floor was laid, on which the water-proofing was placed. Between 27th and 33d Streets, a new process of water-proofing was tried. The great cost of the membrane water-proofing called attention to the possibility of using a compound to be mixed with the concrete with a great saving in cost. Therefore a powder compound was mixed with the concrete in this section.

It was anticipated that the shrinkage of the concrete in drying would open up cracks which would require caulking, but the saving over the cost of the previous method was calculated to pay for this and still effect considerable economy.

Tunnel Ducts.—The ducts in the subway for the cables of the power and light services were laid on the toes of the side-walls and under the platforms of the stations. These ducts were laid after the completion of the shells of the subway and stations, and call for little description, as no difficulties were met.

All the ducts were of vitrified clay, usually of 4-hole pieces, 30 in. long, with the addition of 2-hole pieces in certain places. The joints were wrapped with canvas, and two dowel pins were placed in each duct joint. The joints were made with 1:3 mortar. Bond straps of $\frac{1}{8}$ by $1\frac{1}{2}$-in. iron were laid between the ducts to bond the concrete faces to the ducts. Manholes were built in each subway tube, between stations, and at each end of the stations for pulling and splicing purposes. The duct banks were covered with a concrete jacket to protect them from injury, and the faces of this jacket were reinforced with expanded metal or wire cloth.

Sewers.—Previous to the commencement of the subway work, a main outlet sewer, 5 500 ft. long, was built in 18th Street from Sixth Avenue to the North River, commencing at Sixth Avenue as a concrete and brick structure 5 ft. 9 in. in diameter. At Eleventh Avenue it is a box-shaped section, of reinforced concrete, and is finished at the river end with two barrel sewers supported on the timbers of the steamship pier at West 18th Street.

The original sewers in Sixth Avenue were in the center of the street, and, as they were below the level of the roofs of the subway and stations, the entire system, from 12th to 33d Streets, had to be rebuilt. The old sewers were broken away as the work progressed, and the flow of sewage was maintained through temporary 12 to 20-in. spiral riveted pipes. In rebuilding, it was necessary to provide a sewer on each side of the subway, and to connect those in the side streets.

The new system in Sixth Avenue consists of a trunk sewer on the east side, flowing north from 14th Street to the sewer junction under the subway at 18th Street, 3 ft. 6 in. in diameter at 14th Street, and 4 ft. in diameter at the junction; and a trunk sewer from 31st

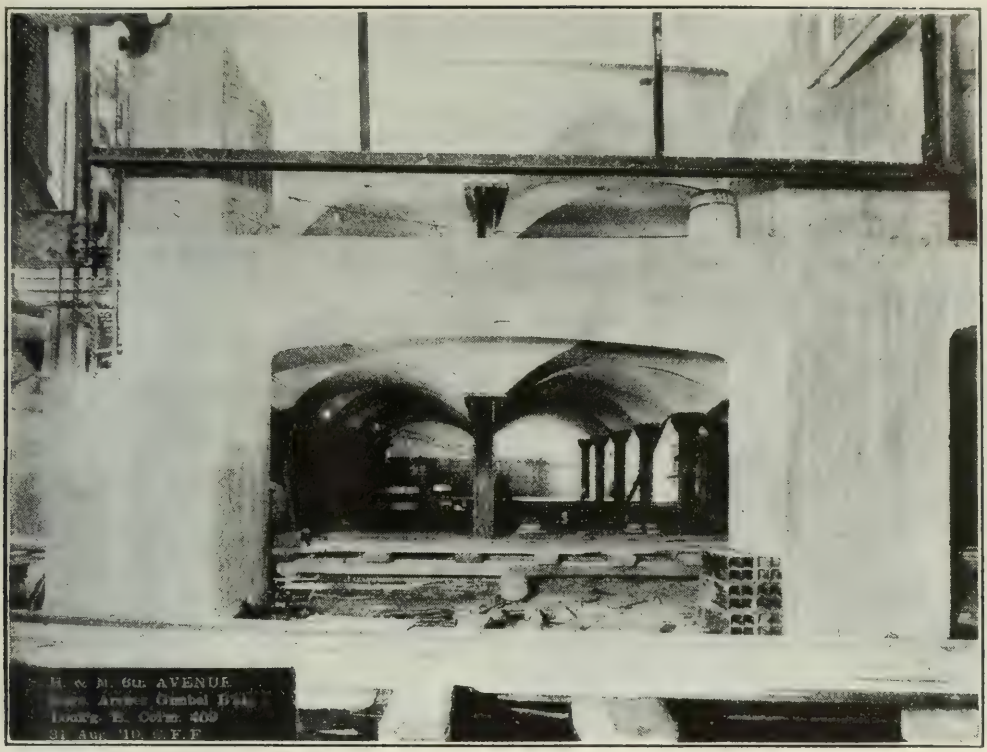


FIG. 19.—TRAIN FLOOR AND CONCOURSE OF THIRTY-THIRD STREET STATION.

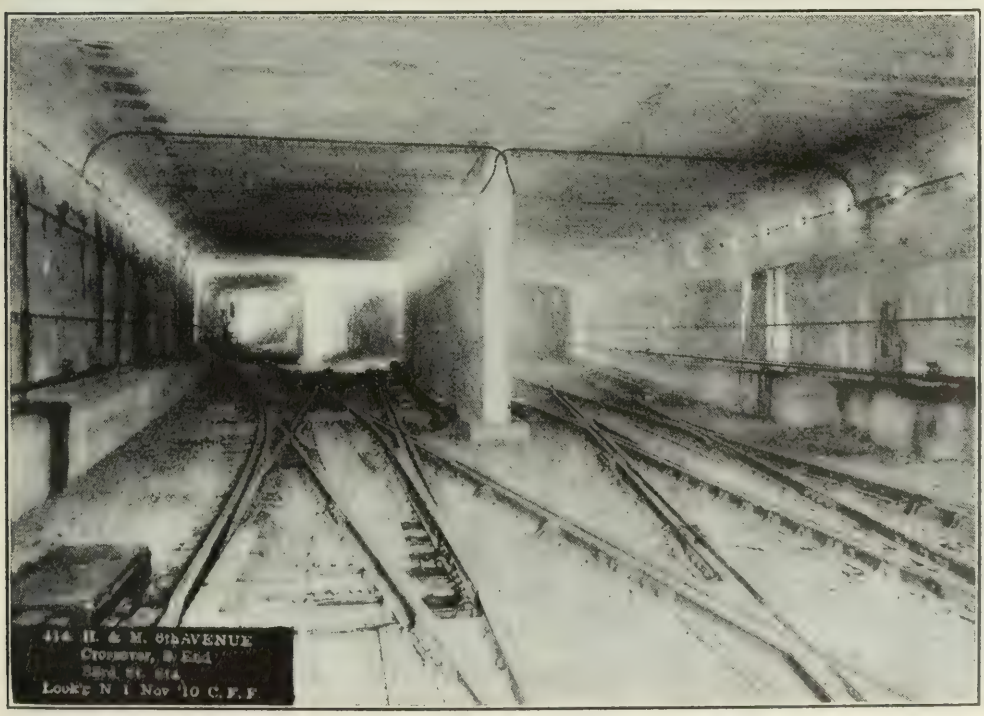


FIG. 20.—ENLARGEMENT OF SUBWAY SOUTH OF THIRTY-THIRD STREET STATION.

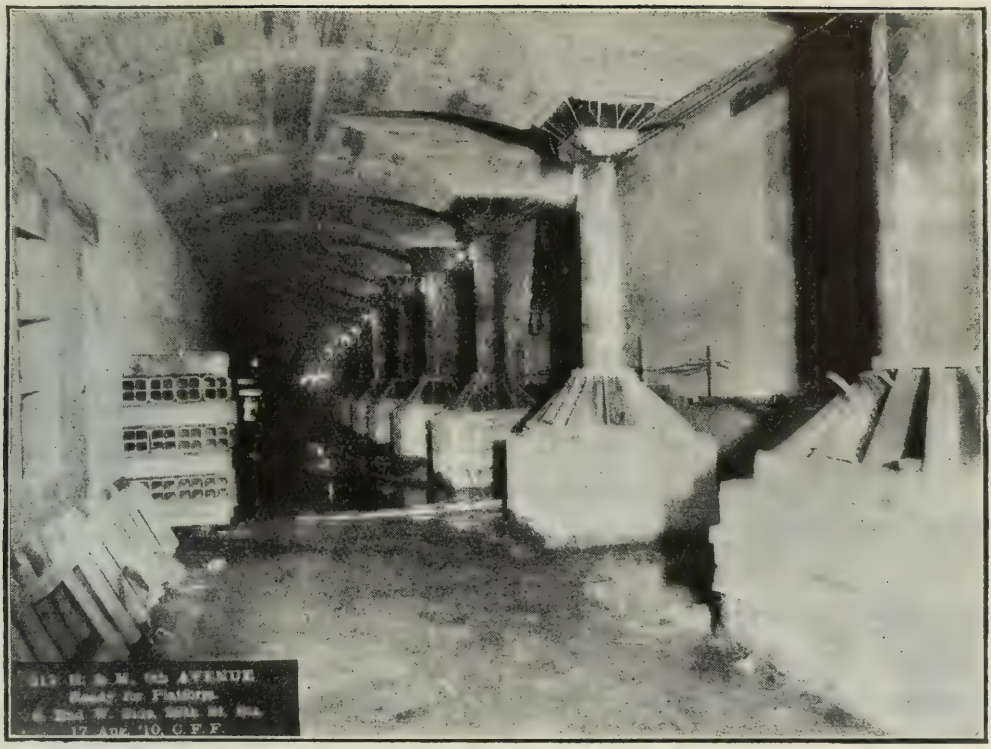


FIG. 21.—STANDARD STATION BEFORE PLATFORM WAS BUILT.

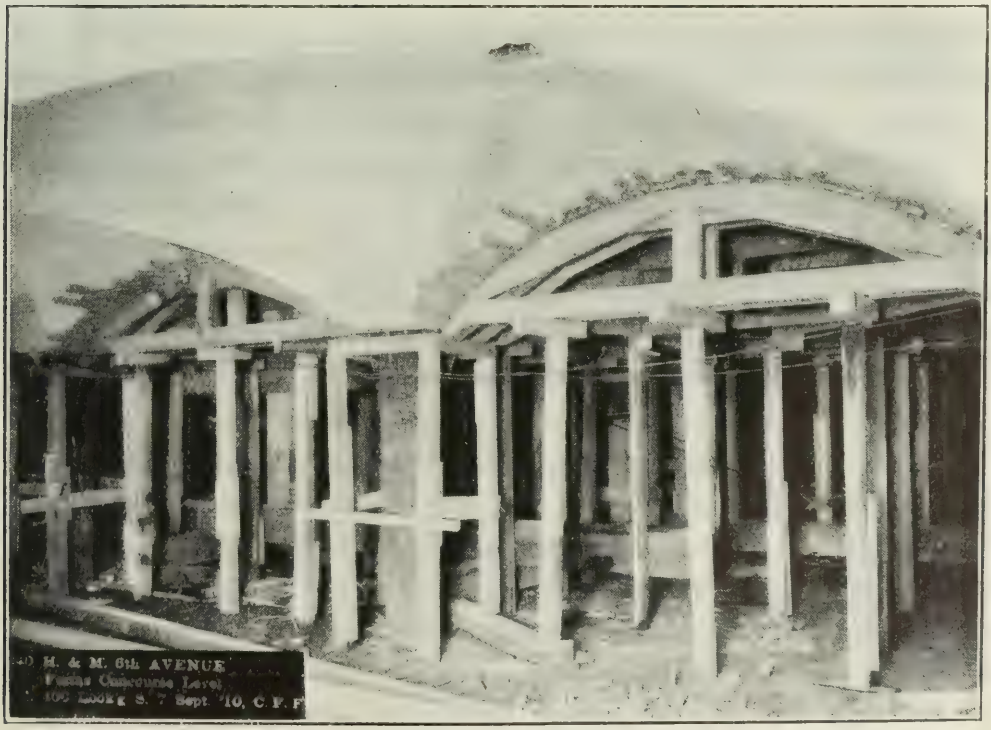


FIG. 22.—FORMS FOR GROINED ARCHES OF ROOF, THIRTY-THIRD STREET STATION.

Street south to the junction at 18th Street, 3 ft. 6 in. in diameter at 31st Street and enlarging to 4 ft. 9 in. at the junction, running down the east side from 31st Street to 21st Street, passing under the 28th and 23d Street Stations and crossing under the subway south of 21st Street to the west side where it turns south to the junction at 18th Street. Smaller laterals, ranging from 12-in. vitrified pipe to 3 ft. 6-in. by 2 ft. 4-in. egg-shaped sewer, were built on both sides of Sixth Avenue at the points not served by the trunk sewers.

The pipe sewers were laid on a concrete invert or foundation 8 in. thick; all the other sewers were built with 8 to 12-in. concrete inverts and 8-in. brick crowns. The forms for the concrete invert were of pine ribs with 1-in. lagging, or were collapsible steel centers. The house connections were of vitrified or cast-iron pipe, and each sewer connection was made with a cast-iron spur.

Surface Railway.—The surface railway track structure was supported on the bracing and posts of the excavation during construction, and brick piers for heights up to 7 ft., or reinforced concrete bents for greater heights, were built up from the subway and station roofs every 5 ft. under each yoke. The tracks were then jacked up to proper grade and the piers finished under the yokes.

Owing to the settlement of the track structure, a large part of the electric duct benches of the surface railway had to be cut away and rebuilt. There were no accidents during the temporary support of the surface tracks excepting a few "blow-outs" of the power cables which were injured because of their proximity to the work.

Street Restoration, Pipes, etc.—At the commencement of the work there were gas mains, water mains, and electric duct banks, including those for the street railway, and all these had to be relaid. There were also additions laid under agreements with the owners, for which the work was compensated.

All the water and gas mains were supported by 10 by 10-in. timber bents where the excavation was carried under them. These bents were erected on solid earth or rock, or on the roofs of the stations, and were built every 6 ft. on both sides of the subway. During the subway work these pipes, duct banks, and cables (where the ducts were stripped from them so that they could be raised outside the lines of the work) were supported by the bracing and posts of the excavation. Before the general excavation was started, 6, 8, and 16-in.

wrought-iron pipe by-passes were laid at the curb lines for the temporary supply of gas, and then the gas was removed from the mains. The empty mains were slung to the timbers and left until they were relaid in their new positions in the street, at which time the house connections were replaced on the mains and the temporary by-passes were removed. The traffic required that only one side of the street be opened at a time in any block.

The pipes were laid on the timber bents, and the duct benches on the rammed fill, excepting those of the surface railway which were laid on the brick or concrete piers and the dry packing laid between the piers.

When the gas and water services were replaced in the mains, the street was back-filled and the temporary pavement laid and maintained until the fill had settled sufficiently for the laying of the final asphalt pavement.

This temporary pavement was of granite blocks which had been removed from the street when the work was commenced.

Storage Yards.

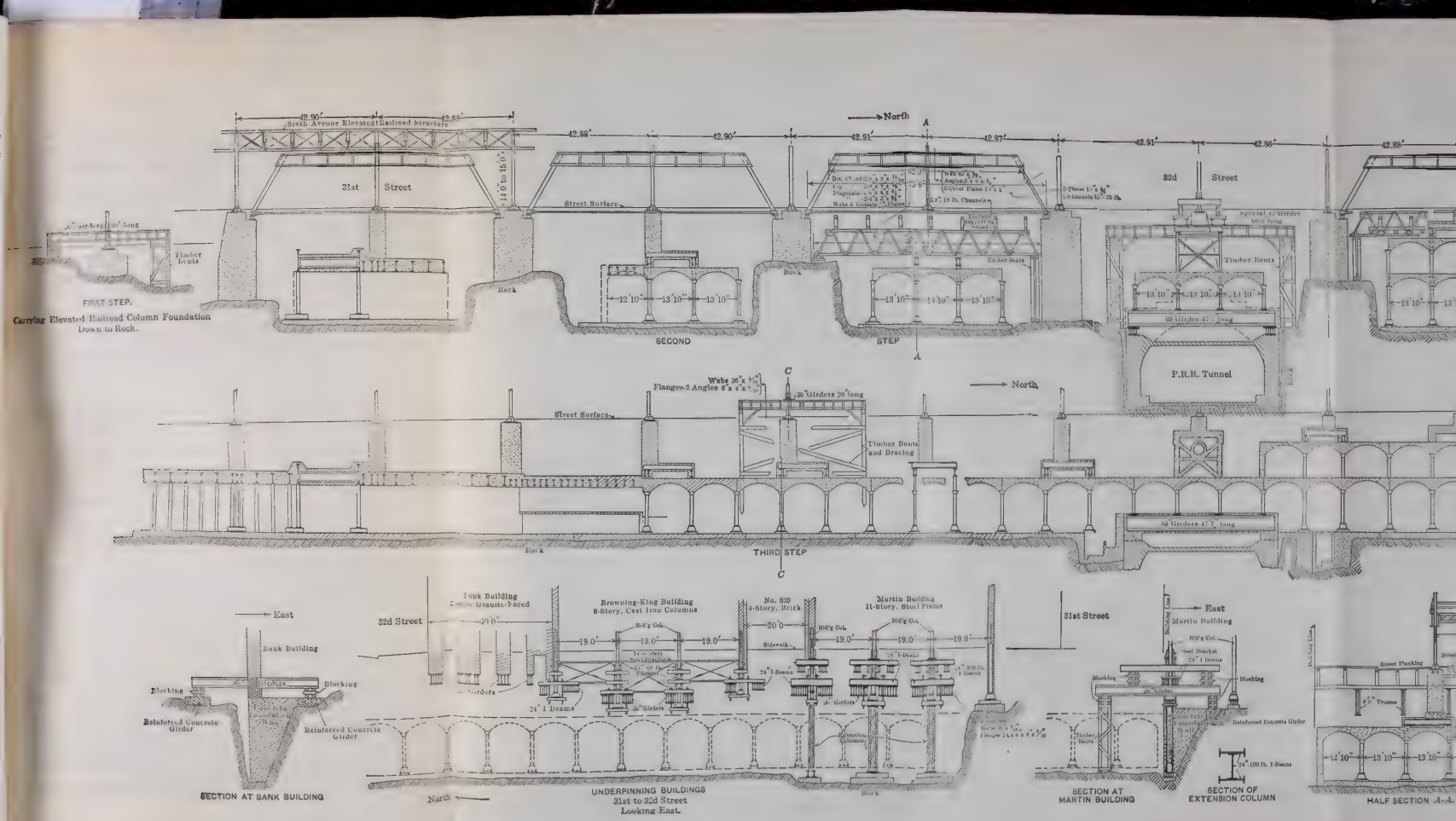
A yard and storehouses were provided at 17th Street and Tenth Avenue for the work between 12th and 27th Streets, and at 42d Street and East River for the work between 27th to 33d Streets. In these yards were the usual storehouses and shops for the storage of materials and plant. The storage of materials in the side streets was limited to 100 ft. on each side of Sixth Avenue. All the cast iron and most of the steel was stored in these yards, in addition to the rods, timber, concrete materials, etc. The two yards were similar, but that at 42d Street contained the power-house for the supply of compressed air.

The haul from the 17th Street yard was from ¾ to 1 mile, and that from the 42d Street yard was from 1½ to 2 miles. The latter yard had the advantage of a dock on the East River at which practically all the materials were delivered, consequently the cost was less than for those delivered at the other yard, which had no dock.

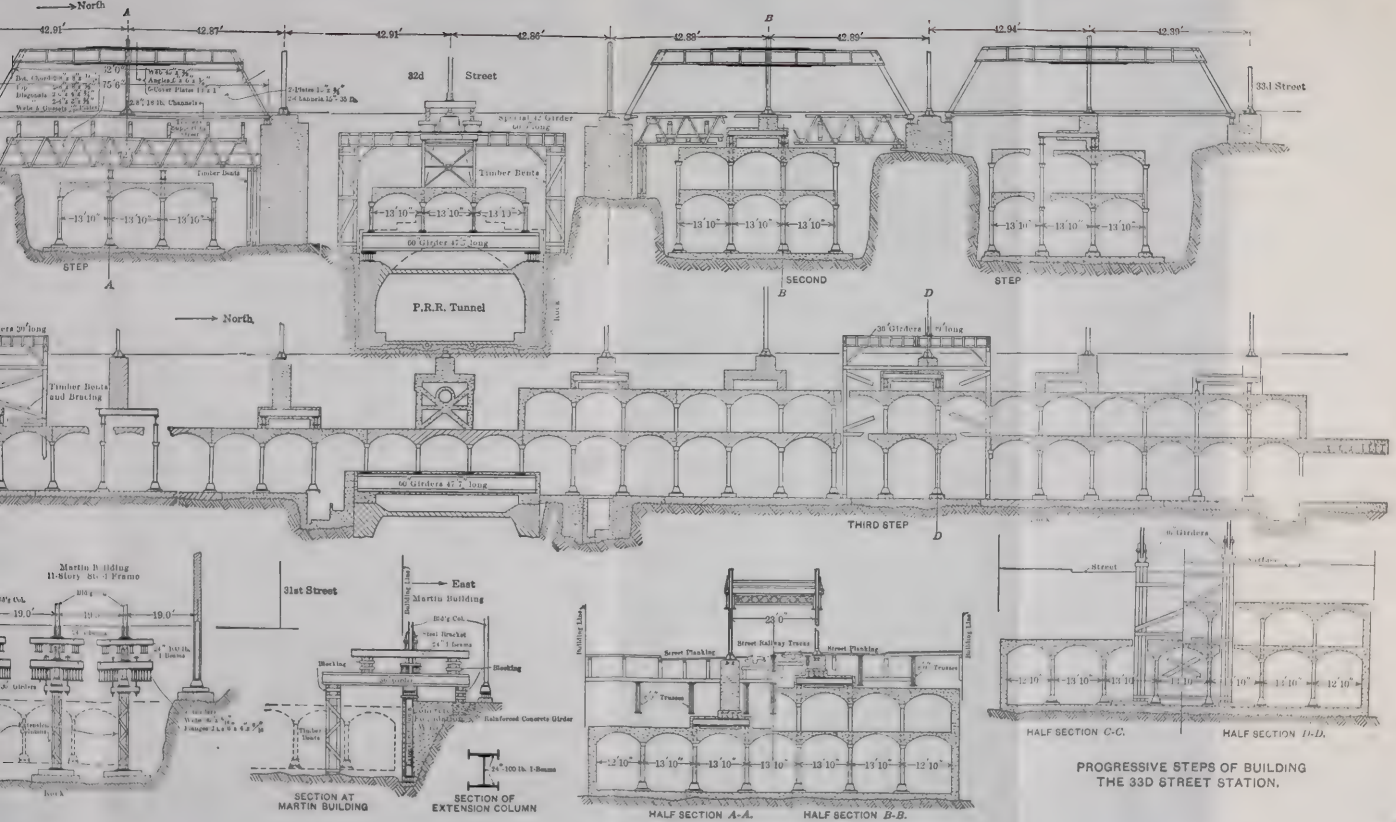
Some of the items which secured economy at the 42d Street yard were as follows:

The use of derricks for unloading material from scows;

The rapid unloading of scows, thus saving demurrage;


Sixth Avenue Elevated Railroad Structure
31st Street
32d Street
Street Surface
Timber Bents
Rock
North
FIRST STEP.
Carrying Elevated Railroad Column Foundation Down to Rock.
SECOND STEP
THIRD STEP
P.R.R. Tunnel
80 Girders 41' 7" long
Timber Bents and Decking
Flanges-2 Angles 6" x 4" x 7/16
East
Bank Building
Blocking
Reinforced Concrete Girder
SECTION AT BANK BUILDING
Bank Building
Browning-King Building
8-Story, Cast Iron Columns
No. 520
1-Story, Brick
Martin Building
11-Story, Steel Frame
Sidewalk
24" I Beams
Timber Bents
Reinforced Concrete Girder
UNDERPINNING BUILDINGS
31st to 32d Street
Looking East.
SECTION AT MARTIN BUILDING
SECTION OF EXTENSION COLUMN
HALF SECTION A-A

5'4A



PROGRESSIVE STEPS OF BUILDING
THE 33D STREET STATION.

The elimination of dockage cost and rentals;

The reliable delivery of the materials in large quantities by scows, as compared with trucking in small quantities.

The greater haul from the 42d Street yard involved a small additional cost over that of the 17th Street yard, but this was more than compensated by the loading of the materials directly from scows into trucks, which, without further delay, could then haul directly to the work.

Besides the storage and loading of materials, the following labor was performed at the yard:

The repair and assembly of plant,

The bending of reinforcing rods to shape,

The building of special forms for the concrete work,

The repairs and assembly of special shapes, etc.

Plant.

The plant was composed of the following main items:

1 power-house equipped with two Ingersoll-Rand compressors having a total capacity of 9 000 cu. ft. of free air per min.;

2 derricks having 25-h.p., Lidgerwood, double-drum engines, operated by compressed air;

8 derricks with Lambert and Lidgerwood 25 and 30-h.p., double-drum, electric engines;

2 cableways with 25-h.p., Lidgerwood, double-drum engines, operated by compressed air;

2 cableways with Lambert, 25-h.p., double-drum, electric engines;

8 concrete mixers—$\frac{3}{4}$-yd., Ransome—operated by compressed air;

26 pumps—Cameron and Worthington—operated by compressed air;

13 electric pumps—centrifugal;

40 rock drills, $2\frac{1}{4}$ and 3-in., operated by compressed air.

The cableways spanned one city block, or from 240 to 260 ft. between towers, and the derricks were all stiff-legged, with masts and booms of such sizes as to suit the conditions at the points of operation. All riveting hammers used for the steelwork were operated by compressed air. The power-house, supplying all the compressed air for the work, was in the yard at 42d Street and East River, and was that previously used for driving the Belmont Tunnel.

The curves on Figs. 25, 26, and 27 show the relative costs of the different items of the construction. The cost of plant is not included. The installation and operation are included in the different items of cost.

The value of the plant was estimated as follows:

Electric and compressed air hoist engines, new and old	$39 400
Concrete mixers	6 520
Pumps—new and old	4 980
Drills—new and old	6 550
Depreciation of power-house during use on this work	15 000
Frames and cables and supplies—new and old	7 000
Total estimated value of plant, as furnished on the work	$79 450

Other items usually considered as plant are included in the different cost curves.

The progress of the excavation, concrete work, etc., at various times is shown by the curves on Fig. 27.

The construction was carried on in two sections, the first (between 12th and 27th Streets) was completed before the commencement of the upper one, between 27th and 33d Streets.

The section between 12th and 27th Streets was commenced in September, 1906, and completed in September, 1909. The section from 27th to 33d Street was commenced in August, 1909, and completed in December, 1911.

The cost of the engineering on the work (exclusive of the main office, which is not included in the curves) was 2.78% of the construction.

Statistics.

The work was commenced in October, 1906, and was finished in November, 1911. In two portions of the work—between 12th and 24th Streets, and between 27th and 33d Streets, including the 33d Street Station—the time required from the commencement of the excavation to the operation of trains in the subway was 14 months, which is considered rapid construction. On the whole work there were only 5 deaths due to accidents of any character.

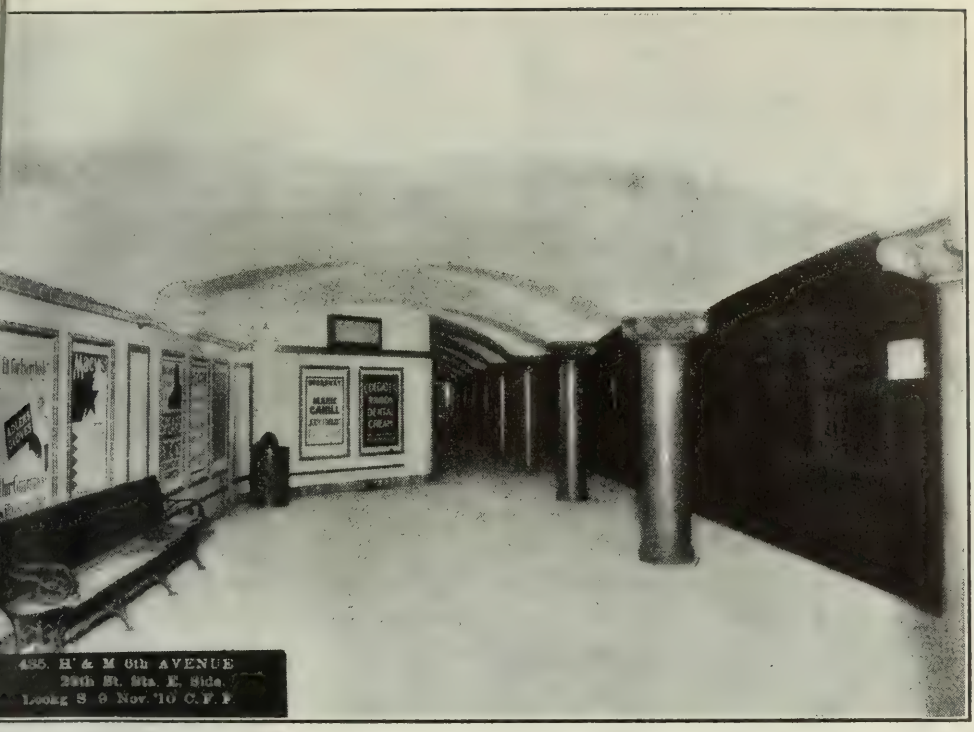


FIG. 23.—FINISHED PLATFORM IN STANDARD STATION.

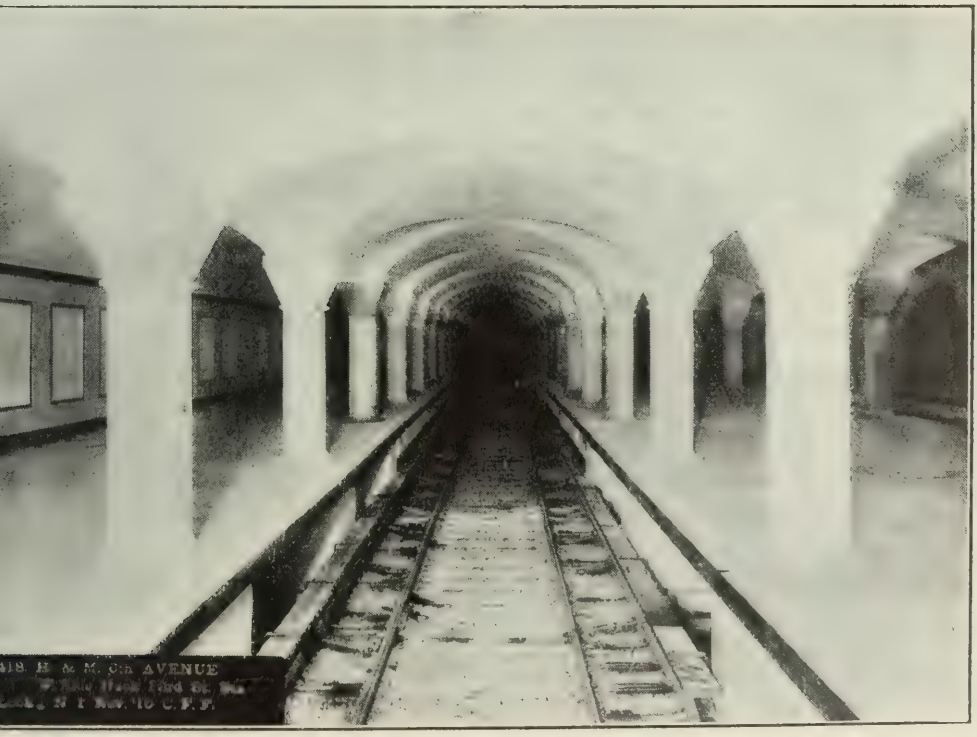
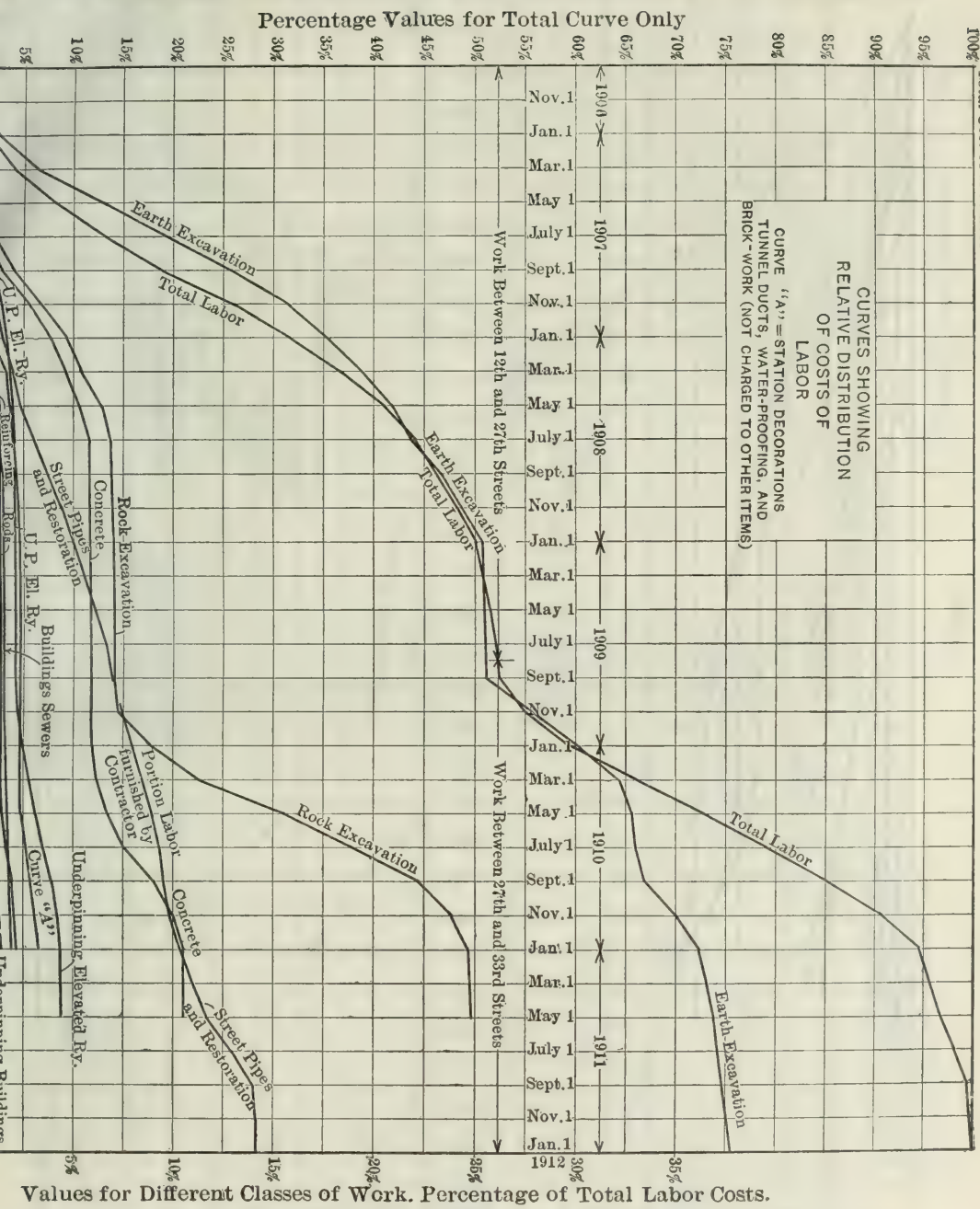
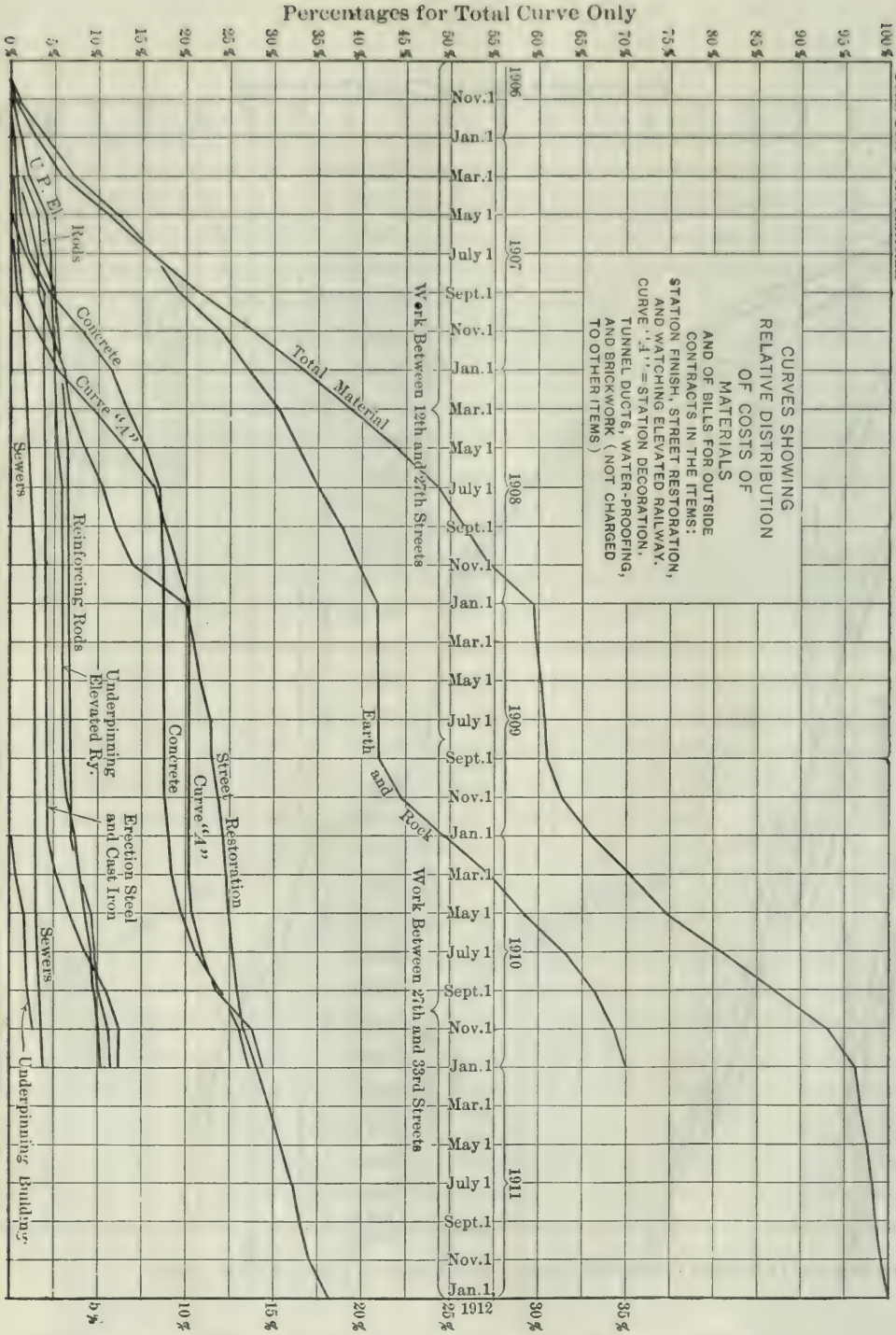


FIG. 24.—COMPLETED PLATFORMS AND TRACK IN THIRTY-THIRD STREET STATION.

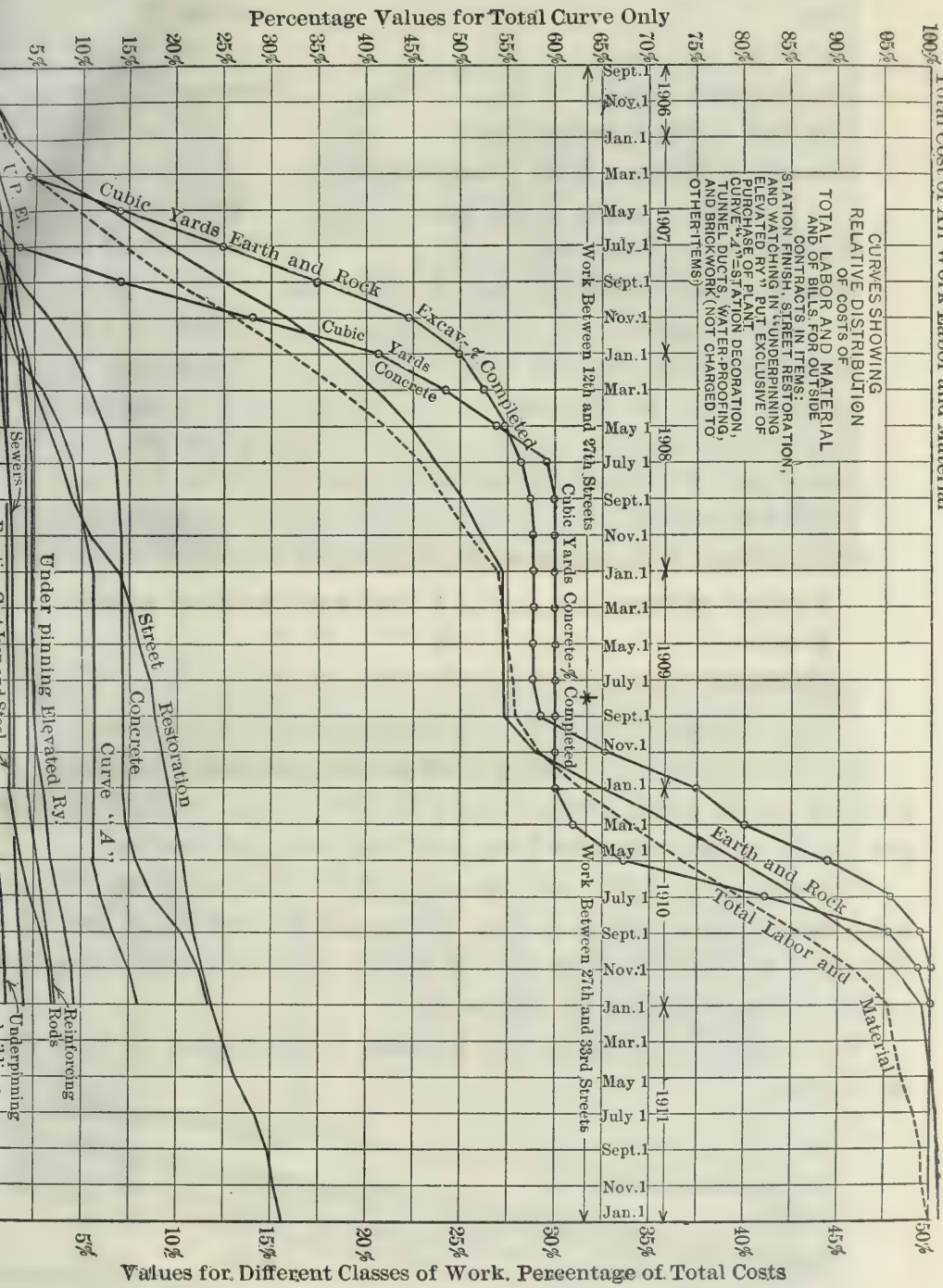


Values for Different Classes of Work. Percentage of Total Labor Costs.



Values for Different Classes of Work. Percentage of Total Material Costs.

FIG. 26.


Percentage Values for Total Curve Only
Total Cost of All Work-Labor and Material
CURVES SHOWING RELATIVE DISTRIBUTION OF COSTS OF TOTAL LABOR AND MATERIAL AND OF BILLS FOR OUTSIDE CONTRACTS IN ITEMS: STATION FINISH, STREET RESTORATION, AND WATCHING IN "UNDERPINNING ELEVATED RY." BUT EXCLUSIVE OF PURCHASE OF PLANT CURVE "A"=STATION DECORATION, TUNNEL DUCTS, WATER-PROOFING, AND BRICKWORK (NOT CHARGED TO OTHER ITEMS)
Work Between 12th and 27th Streets
Work Between 27th and 33rd Streets
Cubic Yards Earth and Rock Excav.-% Completed
Cubic Yards Concrete
Cubic Yards Concrete-% Completed
Earth and Rock
Total Labor and Material
Under pinning Elevated Ry.
Street Restoration
Concrete
Curve "A"
Sewers
Reinforcing Rods
Underpinning
Values for Different Classes of Work. Percentage of Total Costs

The total quantities of the different kinds of work were:

Earth excavation	220 900	cu. yd.
Rock "	95 350	" "
Concrete	65 180	" "
Reinforcement rods	7 090 000	lb.
Elevated railway underpinned	277	columns.
Sewers of different sizes (exclusive of the 18th Street main outlet sewer)	9 500	lin. ft.
Electric ducts in subway	278 500	duct feet.
Miscellaneous brickwork, not included in other items	5 200	cu. yd.
Structural steel and cast iron	2 834	tons.
Interior finish of stations	5	stations.
Street restored	5 400	lin. ft.
Buildings (from 4 to 11 stories) underpinned.	340	lin. ft.
Field engineering	5	years.

The loadings of the street were estimated to be:

Surface railway	7½	tons per lin ft. of street.
Roadways	6½	" " " " " "
Sidewalks	3	" " " " " "
Total	17	tons per lin. ft. of street.

The total quantity of timber used in the excavation, including the planking for the street surface, was 7 703 000 ft. b. m., or 24.36 ft. b. m. per cu. yd. of excavation of both earth and rock, unclassified.

Between July, 1909, and August, 1910, there were 30 700 drill holes blasted, each hole requiring an average of 1.20 lb. of dynamite. The average quantity of dynamite for all rock excavated from 12th to 33d Streets was 0.58 lb. per cu. yd.

The proportions of labor and material for each kind of work was as follows:

	Percentage for labor.	Percentage for material.
In earth excavation	67.78	32.22
In rock excavation	72.95	27.05
Concrete	49.70	50.30
Reinforcing rods	29.98	70.02
Underpinning elevated railway	51.64	48.36
Sewer construction	57.81	42.19
Structural steel and cast iron	21.36	78.64

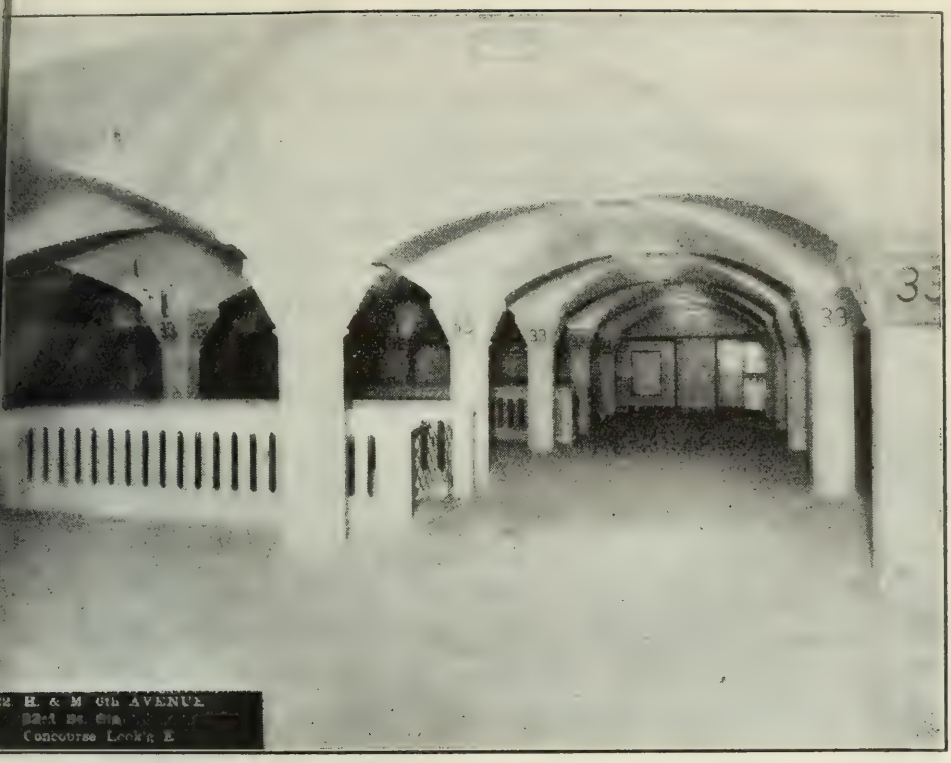


FIG. 28.—VIEW ACROSS CONCOURSE FLOOR OF THIRTY-THIRD STREET STATION.

29.—VIEW ACROSS THIRTY-THIRD STREET STATION SHOWING TRAIN PLATFORMS.

In all the other items of construction there were included in the "Materials" bills for both labor and materials, and the relative proportions for labor and materials were not ascertained.

The average depth of the holes drilled in rock was 14 ft., and the average depth of hole per drill per day was 35 ft.

The proportion of reinforcement rods (in that portion of the concrete which was reinforced) was 106 lb. per cu. yd. of concrete.

The proportions of the different items of cost of construction were as follows:

	Percentage of total cost.
Earth excavation	31.32
Rock excavation	19.16
Concrete (exclusive of rods)	11.74
Reinforcement rods	3.50
Underpinning elevated railway	4.55
Sewer construction	1.69
Tunnel ducts (electric)	0.52
Brickwork (portion not included in other items)	1.35
Structural steel and cast iron	3.34
Station, interior finish	3.29
Street restoration (including relaying pipes, etc.)	15.66
Water-proofing	1.23
Underpinning buildings	0.98
Four entrances through private property	1.67
	100.00

Charles M. Jacobs, M. Am. Soc. C. E., was Chief Engineer and J. V. Davies, M. Am. Soc. C. E., was Deputy Chief Engineer until August 1st, 1909, when Mr. Jacobs became Consulting Chief Engineer, Mr. Davies, Chief Engineer, and James Forgie, M. Am. Soc. C. E., Deputy Chief Engineer. G. D. Snyder, M. Am. Soc. C. E., was Principal Assistant Engineer. The writer was Resident Engineer in direct charge of the work, and Mr. O. J. Theiss and Leslie Muller, Assoc. M. Am. Soc. C. E., were Assistant Engineers in charge of the work in the field. Mr. William Langston was Chief Inspector.

The Degnon Contracting Company were the contractors, for whom H. C. Sanford, Assoc. M. Am. Soc. C. E., was Chief Engineer and R. P. Gustin, Assoc. M. Am. Soc. C. E., was Engineer in Charge. Mr. M. J. Morris was Superintendent.

DISCUSSION

Mr. Whitney.

T. B. WHITNEY, JR., M. AM. SOC. C. E. (by letter).—Judging from the author's statement that the weight of the reinforcing steel per cubic foot of concrete is excessive, it might be inferred that some abnormal percentage of steel was used in the design of the reinforced concrete in this structure. However, the steel ratio is not more than three-fourths of 1%, except in the side-walls which required reinforcing in both faces, where it amounts to 1.20 per cent. The longitudinal bars used to prevent shrinkage cracks averaged about fifteen-hundredths of 1 per cent. A minimum spacing of 6 in. from center to center was used for the steel in order to facilitate the pouring of the concrete.

The arched roof formed a characteristic feature throughout the underground structures of the Hudson and Manhattan Railroad, and this led to its consideration in the design of the subway section on Sixth Avenue. The adopted section consists of a double, arched roof supported by a center wall and side-walls. The side-walls are designed as reinforced concrete slabs to carry the earth pressure as well as the thrust of the arches, and are tied together at top and bottom by tie-bars in the roof and floor.

A comparison of this subway section with two other well-known subway sections shows that the quantities of excavation, steel, and concrete, are slightly in its favor, when the sections are reduced to the same dimensions, using the same loading and unit stresses in each case. The economic features may be compared by referring to Fig. 30. It will be noticed that the New York Subway has the smallest side clearance and that the steelwork is calculated on a basis of a unit stress of 20 000 lb., which accounts for the smaller quantities.

The permanent structure was designed to conform with the live and dead loads as given by the specifications of the New York Public Service Commission, though lower unit stresses were used than are prescribed by these specifications.

In each group of girders spanning the Pennsylvania Tunnel at 32d Street, the girder adjacent to the track carries half the trough track floor and train loads, in addition to its equal share of the permanent overhead loads, consisting of station platform, columns, roof, 20 ft. of fill, street railway, street loads, and elevated railroad structure. Economy of space was essential, so the over-all dimensions of all the girders were kept the same, and the additional flange area required for the girders carrying the track floor and train loads was provided by extra sets of angles placed inside the main flanges of the girder. The increase in web thickness, to provide proper rivet spacing, gave a surplus of material sufficient to carry the additional shear, as

Mr. Whitney.

MARKET ST. SUBWAY, PHILADELPHIA

HUDSON & MANHATTAN R.R.CO.

NEW YORK RAPID TRANSIT SUBWAY

QUANTITIES PER LINEAR FOOT OF TWO-TRACK SUBWAY

	Inside Clearance	Outside Width over all	Excavation, in Cubic Yards	Concrete in Cubic Yards	Reinforcing Steel in Pounds	Structural Steel in Pounds	Total Steel in Pounds	Water-proofing in Square Yards	Concrete over Water-proofing in Cubic Yards
Hudson & Manhattan	13′0″	31′7″	33.90	5.67	820	—	820	10.42	1.50
N.Y.R.T.Subway f=20,000 lb.	12′3″	29′10″	31.80	4.10	—	816	816	9.78	1.47
N.Y.R.T.Subway f=16,000 lb.	12′3″	29′10″	31.80	4.10	—	991	991	9.78	1.47
Market St.Subway Philadelphia	13′3″	37′10″	43.70	7.80	574	206	780	3.50	0.29

FIG. 30.

Mr. Whitney.

shown by Fig. 31. Thus, the extremely heavy overhead loads are carried by the main body of the girder, while the comparatively light track floor and train loads are carried by the inner sets of flange angles.

The trough track floor rests on the outstanding leg of the inner flange angles, which is braced by stiffener angles, 12 in. from center to center, these angles transmitting the loads to the web of the girder. This arrangement simplifies the renewal of these floors.

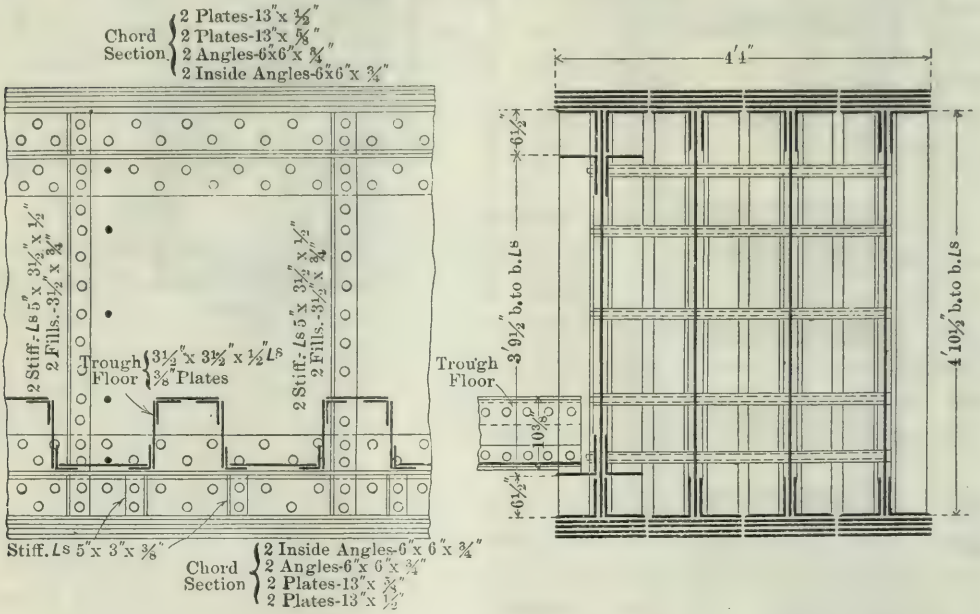


FIG. 31.

Mr. Boucher.

WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—While in Chicago and associated with the organization planning the proposed subway system for that city, about 2 years ago, the matter of water-proofing was quite thoroughly investigated by the speaker. Having been connected with the construction of the New York Rapid Transit Railroad, he was well acquainted with the membranous method of water-proofing, but the integral method of mixing a powdered or liquid substance with the concrete had not been examined to any extent.

At least six manufacturers of compounds offered, and had tests made of, their products, among which were McCormick, Medusa, Truss-Con, Hercules, Ceresit, and Hydrolithic. Due to various causes, the tests were not in every way satisfactory, but it was learned that laboratory tests were of little or no value. No subway has yet been built in Chicago, but the street railway tunnel under the Chicago River at La Salle Street has now been in use about a year. The reinforced concrete approaches to this tunnel, and the concrete-lined steel tubes forming the river crossing are water-proofed solely by mixing McCormick compound with the concrete. The speaker has

Mr. Boucher.

CROSS-SECTION OF ORIGINAL REINFORCED CONCRETE SUBWAY
ON LINE OF RAPID TRANSIT RAILROAD, LENOX AVENUE AND 141ST ST., NEW YORK.

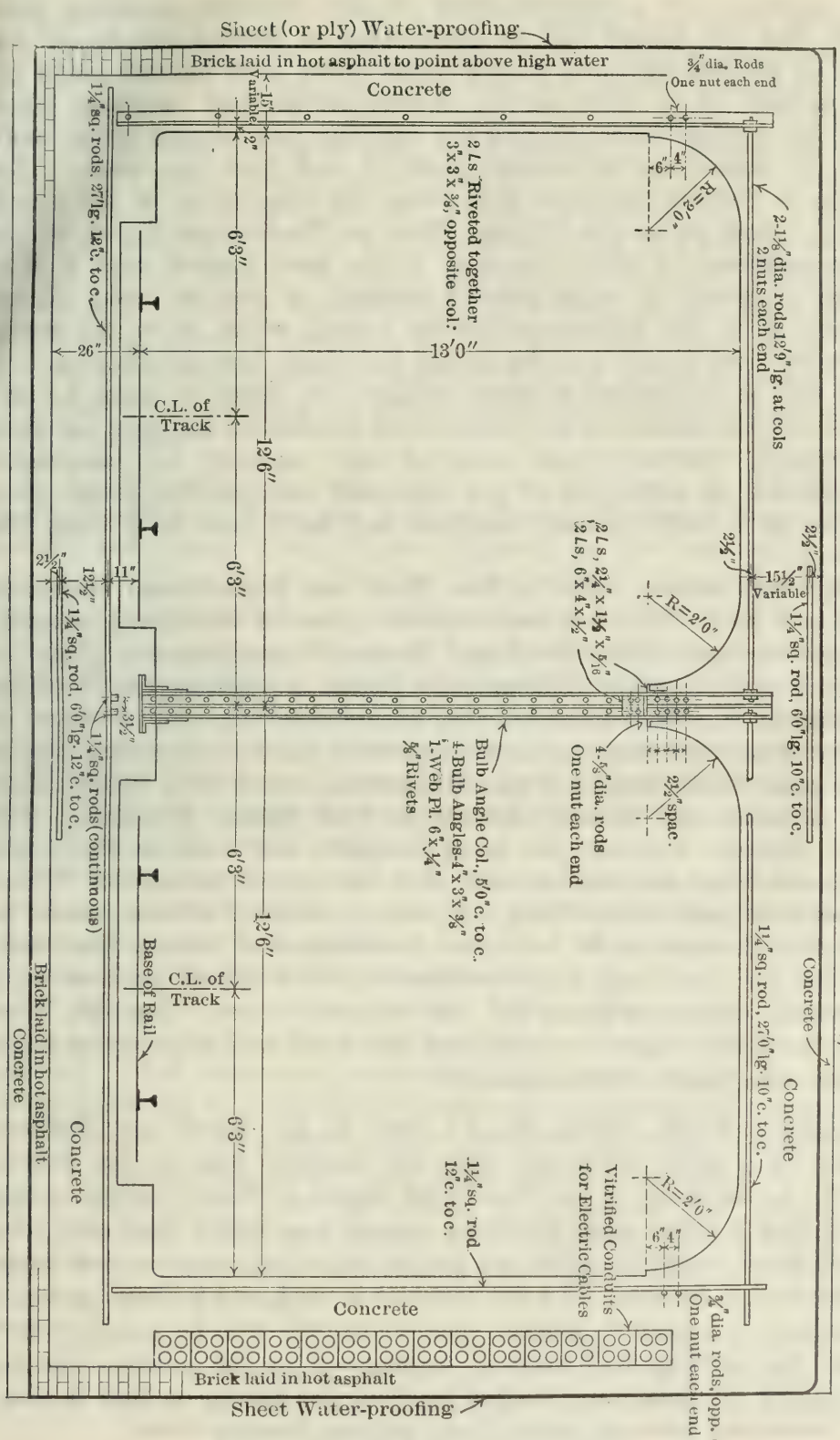


Fig. 39

Mr. Boucher.

been informed by a representative of the company handling that brand, that the tunnel and its approaches at present are as dry as could be desired, the only places where water ever entered being at terminations of a day's work. These cracks and openings were closed by drilling holes through the concrete and forcing grout composed of water-proofed cement back of and into the mass. The speaker is further informed that since the completion of the street railway tunnel under the Chicago River at Washington Street, which was constructed of plain concrete, it has been treated with a thin mortar or grout of water-proofed cement, at various leaky places (as well as in the pump-room of the tunnel, which is on the shore, but below river level), and the results have been very satisfactory.

It will be interesting to learn whether the result of using a compound with the cement in the Hudson and Manhattan Railroad has been satisfactory, whether cracks occurred and caulking was necessary, and whether the extra cost of the compound was justified, rather than the use of a richer cement mixture and more care in placing the concrete.

The first subway built in New York was a steel-beam structure, walls and roof. The first experimental piece of reinforced concrete subway was built from the Rapid Transit Commissioner's plans on Lenox Avenue, northward from 141st Street, a cross-section of which is shown by Fig. 32. Then followed an era, covering about 6 years, in which the subways were mainly of reinforced concrete, this construction being used under Battery Park, Manhattan, and Fulton Street, Flatbush Avenue, and Fourth Avenue to 40th Street, Brooklyn. The newer subways, however, are being designed and built as steel-beam structures, which seems to indicate that this type is preferable. Where surface cars and vehicle traffic, as well as elevated columns, must be supported, it seems to be far better to construct of beams, than rods, for then the loads may be transferred to the steelwork as soon as it is riveted, without waiting for the concrete to set. Can Mr. Burrowes state the reasons for adopting the reinforced type of construction for the Sixth Avenue subway?

Mr. White.

Lazarus White, Assoc. M. Am. Soc. C. E.—Would not Portland cement be a better compound than the material used on the subway work? In the work of the New York Board of Water Supply it was found that a little more Portland cement was better than any compound which could be added, no matter what its name—a mere name not producing dryness. If a wet concrete is used, and a liberal quantity of cement is carefully placed, that compound will be nearly water-proof; but nothing will make it water-proof at the joints. The advantage of the membrane method used on the subway is that it spans the joints and prevents water from getting through them.

Mr. White.

Several tunnels were built for the Catskill Aqueduct without any water-proofing other than the liberal use of Portland cement, and, in the speaker's opinion, these were dry enough to run subway trains through. These tunnels were deep, being several hundred feet below the ground, and after they were grouted off, successfully sustained very heavy ground-water pressures with very little leakage through the body of the concrete. A test* was made at the Wallkill Siphon, in which a stretch of tunnel was subjected to a head of several hundred feet of water. The arch had not been grouted, and leaked very slightly, but there was practically no leakage through the body of the concrete, showing conclusively that Portland cement is a sufficiently good water-proofing. The quantity of cement in this concrete, the speaker believes, was about 1.8 bbl. per cu. yd., corresponding to a 1:2:4 mix.

Mr. Oestreich.

H. L. Oestreich, M. Am. Soc. C. E.—The roof of the Fourth Avenue Subway, in Brooklyn, is of reinforced concrete, but no water-proofing was placed, a 6-in. layer of gravel on the roof acting as a drain to carry the water to the side of the tunnel. It was found that, if the structure leaked at all, it would be at the junction of two days' work. It is a question, therefore, whether Mr. Burrowes would not find the same conditions, even if he used the water-proofing powder in the concrete; in other words, while the slab placed one day might be water-proof, the line at which it joined the next day's work would show a leak. That was the experience on the Brooklyn Subway. It seemed almost as though the concrete formed a better bond with any other material than with itself.

In order to prevent leakage from the roof at the junction of two days' work, a furrow about 1 in. deep and 2 in. wide was cut into the concrete along this line and filled with tar. This helped the process of silting up, and the roof became practically dry.

On two contracts of the Fourth Avenue Subway the brick in mastic was omitted on the sides and bottom of the subway, a 6-in. layer of 1:2:4 reinforced concrete being substituted. The reinforcing consisted of ¾-in., longitudinal rods occupying four-tenths of 1% of the area of the concrete, the intention being to cause the shrinkage to be in many and small cracks rather than in one large one. Overlapping the rods 48 diameters tended to strengthen the junction of two days' work. Constant pumping for 2 years has reduced the elevation of the ground-water temporarily, but it is probably too early to speak definitely of results.

Mr. Burrowes.

H. G. Burrowes, M. Am. Soc. C. E. (by letter).—In regard to the water-proofing, the membranous method had been used between 12th and 27th Streets, and had not given entire satisfaction, as leaks had

* *Engineering News*, December 14th, 1911.

Mr. Burrowes.

developed after completion; therefore, in order to obtain better results, Keystone Water-Proofing Compound was used experimentally between 27th and 33d Streets. In general, it was satisfactory, except at stations, because a leak which would be unnoticed between stations, would be very unsightly and objectionable at a station.

As it was necessary to construct the subway in alternate slices or sections, first under the elevated column bents and then between these bents, there were numerous joints, and it was there that the leaks occurred, and also at places where structural steel extended through the concrete or where reinforcing rods ended close to its inner face. These leaks were stopped by cutting away several inches of concrete from the inside and then plastering with cement mixed with water-proofing compound, by grouting, and, in some cases, where the back-fill had not been completed, by stripping the roof and then placing brick laid in hot asphalt.

The cost was much less than with the membranous method, for not only was the cost of the compound less than that of the membrane, but the outer protecting masonry and the additional excavation for it became unnecessary; even by adding the amounts spent on stopping leaks, there was an ultimate saving.

It is very doubtful whether better results could have been obtained by the use of additional cement, as the leaks were due to the joints in the concrete between sections as built and to the presence of steel which extended through or nearly through the concrete roofs.

Under the practical conditions that prevail in executing such work, it is probable that some leaks would have occurred, no matter what class of water-proofing had been used. The conclusion reached was that the use of the compound was satisfactory between stations, but that at the stations the water-proofing results were not positive enough.

The decision to use reinforced concrete instead of structural steel was made for the reason that there is a slight saving in quantities due to the use of the former, as stated by Mr. Whitney; and, in addition to this, the unit cost of the rods was less, and at the time that this structure was designed, prompt delivery of structural material could not be obtained, while rods could always be had.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1240

MAXIMUM STRESSES IN BASCULE TRUSSES.

By W. Watters Pagon, Jun. Am. Soc. C. E.*

To the best of the writer's knowledge, nothing has been published on the subject of the maximum dead stresses in the trusses of bascule bridges, of the trunnion, rolling lift, or combination types, and, as a result, several instances have come to his notice where the stress sheet showed only the dead stresses for the bridge closed, and not the maximum stresses. In one case the true stress was three times the stress

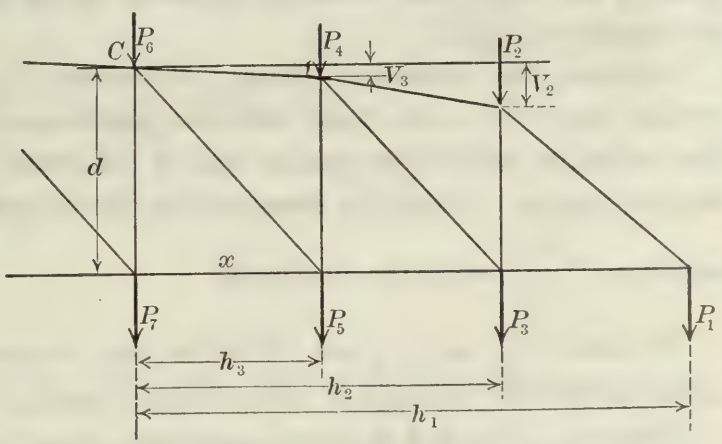


Fig. 1.

shown, though, fortunately, in this particular member, this was not the governing stress. There were several members, however, where the dead stress was the governing one, and in some of these the section had to be increased about 6 per cent.

Method of Determining Maximum Stresses.—Assume the truss shown in Fig. 1, subjected to the joint loads, P_1, P_2, P_3, etc.; the

* Now Assoc. M. Am. Soc. C. E.

problem is to find the maximum dead stress in the lower chord member, x. Let the truss revolve about a horizontal axis at O. Find the dead stress in x with the truss in the horizontal position. Call this stress S_H. Find the dead stress in x with the truss vertical, or 90° from the first position. Call this stress S_V. Then the true maximum dead stress is $S = \sqrt{S_H^2 + S_V^2} = \frac{1}{d}\sqrt{M_H^2 + M_V^2}$, where M_H and M_V are the respective moments about the moment center, C. In parallel chord trusses without secondary system, $S_V = (P_1 + P_3 + P_5$, etc.) $\times$ $d \div d$, equals the sum of the bottom chord loads between the member and the free end of the truss; but for parallel chord trusses with secondary system, S_V equals the sum of all the bottom chord and half of the middle joint loads between the member and the free end.

In the foregoing, attention has been devoted to bottom chord members only, for, although the theory applies to all members, the top chords and web members* receive their maximum dead stresses when the truss is in the horizontal position. In any special case, it will be easy to determine which members need to be investigated for maximum stress by the use of the following criterion, which is easy to remember and simple to apply.

Criterion for Maximum Stress.—Whenever S_H and S_V for any member are of like sign, there will be a maximum dead stress, greater than either, to which the member will be subjected during every opening and closing. When this stress occurs, the bridge will be open at an angle $\tan.^{-1}\frac{S_V}{S_H}$, with the horizontal.

Whenever S_H and S_V are of unlike sign, the maximum stress will be one of the two, whichever is the larger (usually S_H).

Summary.—To find the maximum dead stress in any member in a truss revolving about an axis perpendicular to it: Find S_H for the bridge closed. Find by inspection whether S_V will have the same sign as S_H. If of like sign, find S_V and then the maximum stress from $S = \sqrt{S_H^2 + S_V^2}$. If of unlike sign, S_H (or S_V) is the maximum stress.

As the methods and theory given are perfectly general, any two directions at right angles to each other may be substituted for

* For web members this is strictly true only in parallel chord trusses.

"horizontal" and "vertical." They may also be applied to any revolving truss subjected to a number of parallel loads which are not vertical.

Theory.—In Fig. 2, which shows the truss open at an angle, α, let the loads and dimensions be the same as in Fig. 1. To find the stress in x for this position:

$$S' = \frac{M}{d} = \frac{(P_1 \cos.\ \alpha\, h_1 + P_3 \cos.\ \alpha\, h_2 + \ldots\ldots P_2 \cos.\ \alpha\, h_2 + \ldots\ldots \text{etc.})}{d}$$

$$+ \frac{(P_1 \sin.\ \alpha\, d + P_3 \sin.\ \alpha\, d + \ldots\ldots \text{etc.} + P_2 \sin.\ \alpha\, v_2 + \ldots\ldots \text{etc.})}{d}$$

$$= \frac{\Sigma\, P\, h \cos.\ \alpha + \Sigma\, P\, v \sin.\ \alpha}{d} = \frac{M_H \cos.\ \alpha + M_V \sin.\ \alpha}{d}$$

$$= S_H \cos.\ \alpha + S_V \sin.\ \alpha.$$

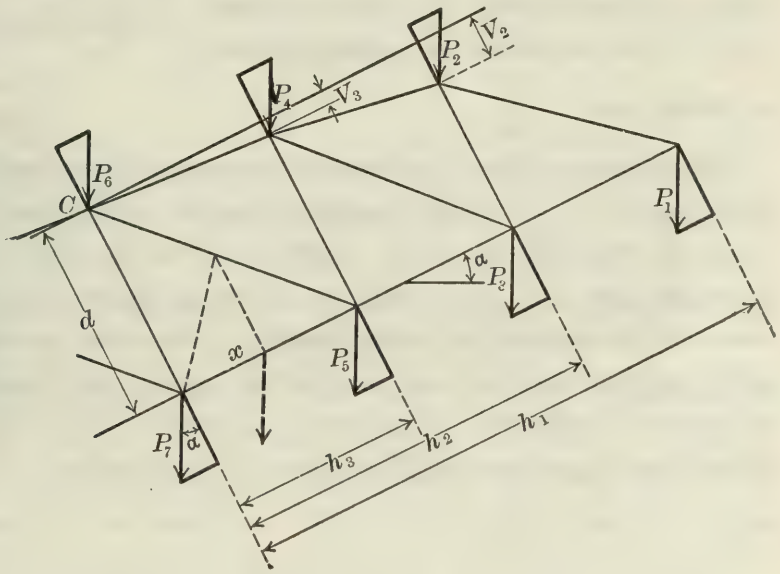


FIG. 2.

In order to find the maximum value of S', differentiate as to α:

$$\frac{dS'}{d\alpha} = -S_H \sin.\ \alpha_o + S_V \cos.\ \alpha_o = 0, \text{ therefore } \tan.\ \alpha_o = \frac{S_V}{S_H};$$

$$\text{and } \sin.\ \alpha_o = \frac{S_V}{\sqrt{S_H^2 + S_V^2}},\ \cos.\ \alpha_o = \frac{S_H}{\sqrt{S_H^2 + S_V^2}};$$

$$\text{therefore, } S = \frac{S_H^2}{\sqrt{S_H^2 + S_V^2}} + \frac{S_V^2}{\sqrt{S_H^2 + S_V^2}} = \sqrt{S_H^2 + S_V^2}$$

= the maximum, S'.

For any member in a secondary system, the same reasoning holds true.

To find a criterion for the position giving the maximum stress, lay off, in Fig. 3, $ab = S_H$, horizontally and measured to the right, if in compression. Then lay off $bc = S_V$, upward if in compression, making abc a right angle. Then $ac = \sqrt{S_H^2 + S_V^2} = S$. Draw a line, ad, at an angle, α, with ab. Project b to e, and c to d. Then $ae = S_H \cos. \alpha$, and $ed = S_V \sin. \alpha$; and, therefore, $ad = S'$ for the angle, α.

Draw a circle through a, b and c. Then, as $abc = 90°$, ac is a diameter, and as adc is a right angle, d lies on the circle. Therefore the length of any chord, ad, at an angle, α, with the horizontal, is the stress, S'. The greatest chord is the diameter, ac, which makes an angle, $\tan.^{-1} \frac{S_V}{S_H}$.

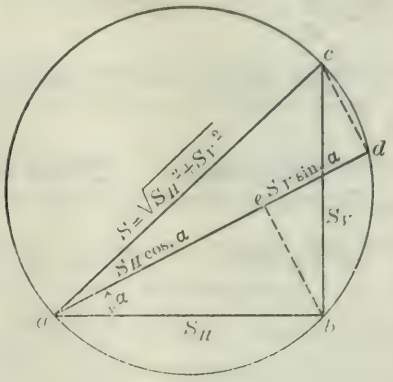


Fig. 3.

This diagram, Fig. 3, completely represents the stress in any bottom chord member. For other members similar circles may be drawn. It is easy to see that for every member having stresses, S_H and S_V, of the same sign, the circle will be drawn as shown, and the maximum stress will be attained at each opening. For other members, the circle will lie in another quadrant, as a is the origin; and therefore this stress will never be attained. Therefore, S_H (or S_V if it is the greater) will be the maximum for which provision is to be made.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1241

STREET SPRINKLING IN ST. PAUL, MINN.*

By C. L. Annan, M. Am. Soc. C. E.

With Discussion by Messrs. S. Whinery, A. H. Blanchard, and C. L. Annan.

"Municipal engineers seem to have always looked on street sprinkling as an every-day, matter-of-course operation, not worth serious attention and study, and not susceptible of much improvement in efficiency and economy. It is safe to say that no other municipal function or process receives so little intelligent attention and supervision."†

For years the citizens of St. Paul put up with unsatisfactory street sprinkling. In the earlier years the work was done by contract, the cost being roughly assessed, at considerable expense, against the property supposed to be benefited. Then, as a substitute, an annual appropriation was included, for a few years, in the general tax levy to meet the cost of sprinkling by day labor. The result was intolerable. Finally, at the general election in May, 1908, a sprinkling amendment was submitted to the people, and in 1909 the Commissioner of Public Works took charge of street sprinkling under this amendment.

The system inaugurated at that time has been fairly perfected, and it has been demonstrated that systematic attention to details has brought results at least commensurate with the cost.

The amendment provided for an appropriation of $60 000 for the first year and an additional $40 000 for the second year. By that

* Presented at the meeting of September 18th, 1912.
† *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 33.

time the system should be smoothly running under the revolving fund fed by the assessment of the property benefited.

The third season under the new law opened with about 230 miles of streets to be sprinkled. New routes were added from time to time and a few were discontinued, resulting in a total gain for the season of about 9 miles. There is generally strong opposition to the discontinuance of a route once started.

From two to eight trips, depending on the nature of the street surface and the amount of traffic, are made each day. There are nine sprinkling districts, each under the supervision of an inspector at $75; a chief inspector or overseer at $110 is also employed. The ten inspectors report daily. The chief inspector advises and assists local inspectors and makes special examinations. The inspectors' reports are tabulated daily; all complaints are investigated, and the written reports are filed for reference. The foreman of stand-pipe construction and repair crews receives daily instructions, and the yardmaster returns a daily statement of sprinkler distribution and repairs. Careful itineraries on each team route are taken from time to time, and, in short, the head of the department is intimately informed of all details concerning equipment and operation.

In assessing the cost of the work, the abutting frontage, number of trips per day, and length of time sprinkled, are taken into consideration. For the season of 1911, 43 assessment rates thus resulted, ranging from less than 1 cent to 11½ cents per front foot of abutting property. The cost of water is not included in the assessment, the City charter so providing.

Water sprinkling of macadam was superseded early in the season by oil spraying. It was estimated that about ⅓ gal. per sq. yd. of a 20% liquid asphalt might be applied three times during the season at a cost not to exceed that of water sprinkling for the same period. The latter part of the season being unusually wet, comparatively few streets required the three coats of oil.

The sprinkling wagons are city property; 40 new and 31 second-hand sprinklers, varying in capacity from 600 to 750 gal., were purchased in 1903. Twelve 750-gal. tanks were added in 1909, and as many more in 1910. These 95 sprinklers allowed a 10% reserve. All were in good repair after a thorough overhauling during the winter.

Teams were used for 7 days in the week at $100 per month, rain

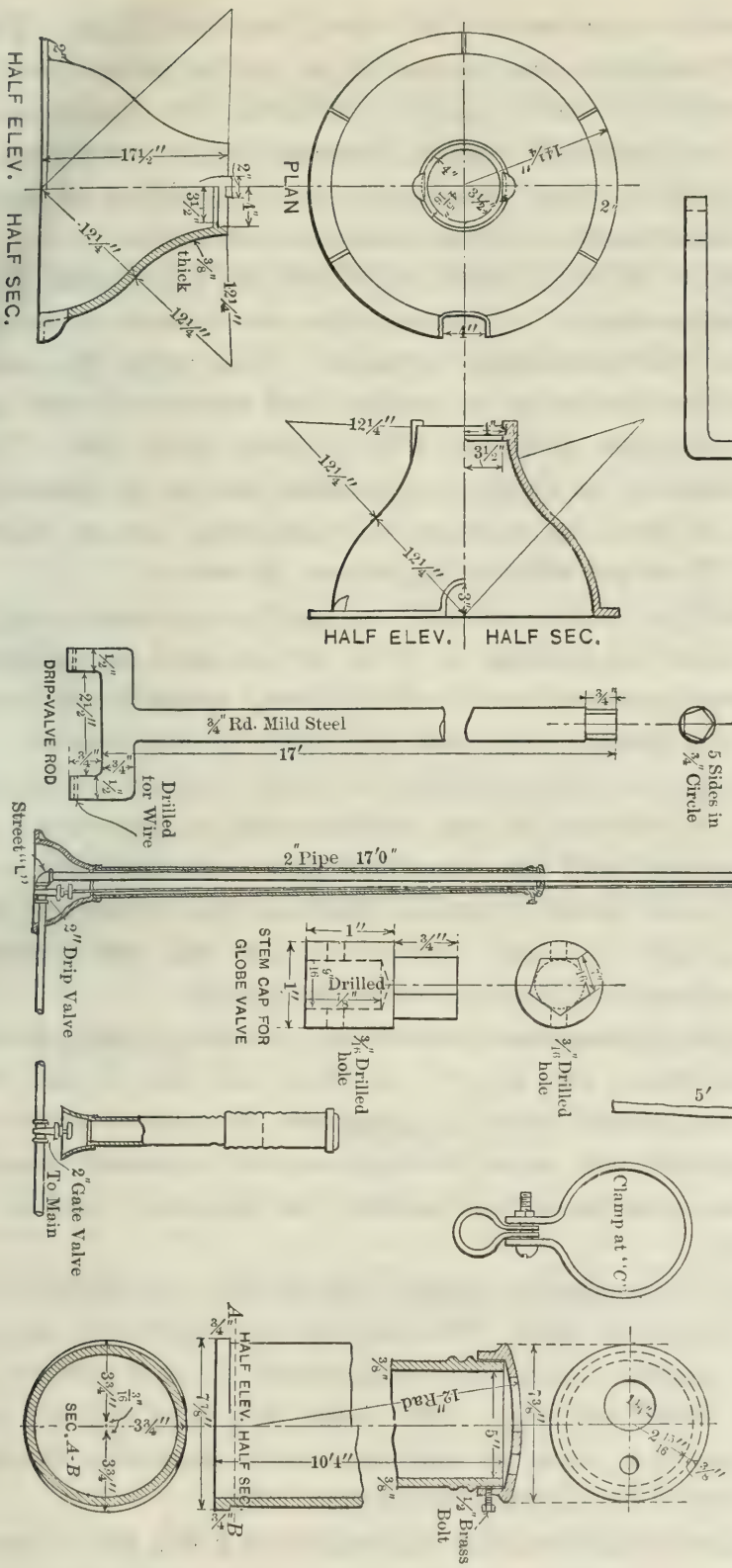

STAND-PIPE AND GATE-BOX
HALF ELEV. HALF SEC.
PLAN
5 Sides in ¾" Circle
WRENCH
HALF ELEV. HALF SEC.
¾" Rd. Mild Steel
17'
5 Sides in ¾" Circle
DRIP-VALVE ROD
Drilled for Wire
2" Pipe 17'0"
Street "L"
2" Drip Valve
STEM CAP FOR GLOBE VALVE
Drilled
⅜" Drilled hole
⅜" Drilled hole
"C"
2" Globe Valve
3'
5'
2" Gate Valve To Main
Clamp at "C"
HALF ELEV. HALF SEC.
SEC. A-B
10'4"
12" Rad.
⅜" Brass Bolt

FIG. 1.

or shine, under an agreement with the Teamsters' Union. Previous to April 1st, teamsters were allowed $4 per day for actual time.

Inspectorships, formerly political sinecures, were transformed into realities. Consistent daily reports, investigations of complaints, route itineraries, and first-hand information as to condition of sprinklers and stand-pipes, require a certain amount of attention.

Five men at $2 and a team at $3 keep the stand-pipes in repair under the supervision of a foreman, who also attends to the maintenance of the public drinking fountains. This force also moved 16 stand-pipes from one point to another and erected 17 new ones in 1911. A stand-pipe, installed, Fig. 1, costs about $60. The top valves are removed in October to prevent damage by freezing, and are replaced in April, an underground valve being used in March and November. There are 463 stand-pipes now in service.

The stand-pipe is quite generally regarded as a nuisance, and much opposition is met in locating it. It is not so much its unsightliness as the frequent presence near it of the heavy sprinkler, the standing horses, and the general sloppiness, that make it objectionable.

For washing ice, mixing cement, and other purposes, the stand-pipe is a great convenience to any outsider with a pipe-wrench. The Water Department and the police had long overlooked this kind of larceny, but a few arrests by special officers of the Sprinkling Department, and a few trials and convictions, now and again, somewhat abated this nuisance of mud and damaging misuse.

The method of assessing is as follows: A clerk, at $60, enters the daily reports (Form 150, Fig. 2) on Form 152, Fig. 3, and indexes complaints and special orders to inspectors by number on the same form. Complaints are taken in duplicate on a general department form, the duplicate being held against the inspector until he turns in his report.

Form 152 is arranged in a check file divided into districts by tab cards of a distinctive color. The districts are subdivided into routes by cards of another color, one compartment in each district being reserved for "dead" trip slips. The current trip slips will not average more than ten to a route, so they are readily accessible, though the aggregate may be considerably more than 1 000.

Trip and route numbers may be changed on a trip slip (Form 152) but a change of limits necessitates new slips and cross-references

Form 150. 12-5-1911 2M.

DEPARTMENT OF PUBLIC WORKS

ST. PAUL, MINN.

DAILY REPORT—STREET SPRINKLING

May 18 1911

District No. 5 Route No. 1 Tank No. 21 Driver Carl Lange

Trip No.	STREET	FROM	TO	Single Trips per day	Length
	Trip Number 1 2 3 4 5 6 7 8 9 10				
	Single Trips 4 4 4 4 4 4 2 2 4 4				
		2	25 G. Petrowske		
	11 12 13 14 15 16 17 18 19 20 21 22 23 24 25				
	4 2 2 2 4 4 4 4 4 4 4 4 4 4				
		3	40 John Bede		
	26 27 28 29 30				
	4 4 4 4 8				
		4 1	26 Fred Wiels		
	31 32 33 34 35 36 37 38 83				
			4 4 4 4 4 4		
31	Indiana from	So. Robert to	Hyde	4 Discontinued	
83	Indiana "	So. Robert "	So. Wabasha	4 Begun	
84	Indiana "	So. Wabasha "	Hyde (Transferred to Route 7)		
	Note: Routes 5–9 inclusive on next sheet				
			Total		

I certify the above to be correct

Inspector

Standard size 7⅛ by 8¼ in.

FIG. 2.

DISTRICT NO. 5. TRIP NO. 82 WARD NO. 6 STREET State

ROUTE NO. 2 FROM Louisa TO Concord

REGULAR TRIPS 4 NUMBER OF TIMES SPRINKLED

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
MAR. APRIL	✓	×	×	×	×	×	×	×	×	×	✓	×	×	✓	✓	✓	✓	2	×	×	✓	✓	✓	✓	✓	✓	✓	✓	✓	2	✓
MAY	2	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	×	×	×	2	×	✓	✓	×	×	×	2	✓	✓	2	×	✓	2	✓
JUNE	✓	×	×	×	✓	✓	✓	2	✓	✓	✓	✓	✓	✓	✓	✓	×	2	✓	✓	✓	✓	✓	✓	✓	×	✓	✓	✓	✓	
JULY	✓	✓	✓	×	×	✓	✓	2	×	✓	✓	✓	✓	✓	✓	✓	✓	×	×	✓	✓	✓	×	×	2	✓	✓	×	✓	✓	✓
AUG.																															
SEPT.																															
OCT.																															
NOV.																															

Form 152 3-21-1911. 3M.
MEMORANDUM NOS. 145

Standard size 8¾ by 3½ in.

FIG. 3.

and the consigning of the obsolete slip to the "dead" division until required for assessment purposes. All assessment forms are handled on the loose-leaf system.

Two men, at $80, are employed from March 15th to August 1st in making a transcript from the County Auditor's records on Form 151 (Fig. 4) and preparing assessment sheets (Form 166, Fig. 5). The force is then doubled to make the assessment, post, and check it, the work to be completed early in October.

The assessment rate, A, per front foot of property abutting on any particular section of sprinkled street is derived as follows:

D = trips per day on the particular route,

M = sprinkling period, in months and tenths, for the same route,

$C = D \times M$ (nearest integer),

F = total frontage on same route,

$P = C \times F$,

S = sum of products, P, for all routes,

T = estimated total cost of sprinkling all routes. (From the five months actual record a close estimate can be made for T.)

$$R = \frac{T}{S},$$

Then $A = C \times R$.

An idea of the method of procedure and of the relation of volumes may be obtained by particular reference to the assessment of 1911. The daily record, on Form 152, was summarized on Form 154 (Fig. 6), embracing 1 439 sheets in twelve groups, one for each ward, each group arranged alphabetically by street names and bound with flat-head fasteners. The coefficient, C, having been determined for each slip, the 1 439 slips were rearranged in 43 groups, corresponding to the variations of C. A five-column table was then prepared with a line for each value of C, thus:

C	F	P	$F \times R$	$C \times R$
1.....	34 006	34 006	62.91	0.00185
2.....	69 252	138 504	256.23	0.00370
		and so on, to		
62.....	62 965	3 903 880	7 222.09	0.11470
	2 567 815	50 529 857	93 480.24	

Form 151. 3-13-1911. 3M

WARD 6 DIVISION 1 Auditors Sub * 30 1290

	FROM	TO	DESCRIPTION	LOT	BLK.	FRONTAGES Feet To Lot	FRONTAGES Totals Feet	FRONTAGES Totals Lots	ASSESSMENT FRACTIONAL	ASSESSMENT TOTAL	CHECK TOTALS
State	Louisa	Concord		33		84			280		
				34		46			153		
				35		35			116		
				36		53			177		
				37		70			233		
				38		76			253		
○				39		100	464	7	333	1545	1545
○											

Standard size 13⅝ by 8½ in.

FIG. 4.

Form 166. 3-13-1911. 2M

720

WARD NO. 6 DIVISION NO. 1 ROUTE NO. STREET State

FROM Louisa TO Concord

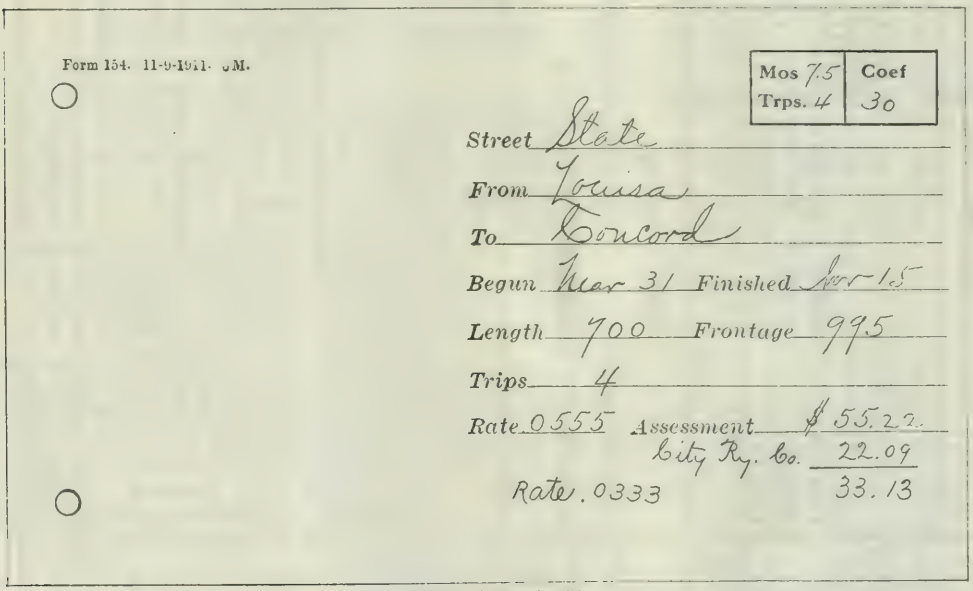
LENGTH 700

	FROM CITY ENGINEER'S RECORD Name of Plat		Frontage	FROM FORM 151 COUNTY AUDITOR'S RECORD Name of Plat	Block	Page	Frontage	No. of Lots	Asst. Rate	AMOUNT Trial	AMOUNT Equated
	W. St Paul Proper	110	271	Auditors Sub * 30		1290	464	7	0.3330	1545	
	" " "	93	110	Louisa Sub	59	1441	150	1		500	
○	Louisa Sub	93	150	W. St. Paul Proper	110	1665	271	3		902	
	Auditors Sub * 30		464	" " " "	93	1648	110	3		366	
			995				995			3313	
○											

The total of Column P is S. The total of Column $F \times R$ is T. R is obviously the first item in Column $C \times R$. R being constant, there are as many values of A as there are of C and $F \times R$, for each group is checked by the total.

The entries on Form 154 are now completed, and the slips are rearranged in ward groups. In the meantime, Form 166 has been prepared to correspond with each sheet of Form 154, of which it is an extension applied to the property to be assessed. Form 166 is arranged alphabetically. When complete, the sheets are paged consecutively from the first sheet of Ward 1 to the last sheet of Ward 12.

The assessment is finally made on Form 151, there being as many sheets of this form as there are block numbers involved on the various plates and subdivisions named on Form 166.


Form 154. 11-9-1911. 3M.

Mos 7.5	Coef
Trps. 4	30

Street State
From Louisa
To Concord
Begun Mar 31 Finished Nov 15
Length 700 Frontage 99.5
Trips 4
Rate .0555 Assessment $55.22.
City Ry. Co. 22.09
Rate .0333 33.13

Standard size 6¾ by 4 in.

FIG. 6.

For instance, on page 720 of Form 166 there are four blocks of property containing lots to be assessed. There will consequently be four corresponding sheets of Form 151. These sheets are arranged alphabetically by plot names in ward groups, and they are also paged consecutively.

Page 1290 (Fig. 4) is a specimen sheet whereon the seven lots in the Auditor's Sub. No. 30, noted on the first line of page 720, are entered in detail. The 1911 assessment contained 3 472 sheets of Form 151.

The cost of making the assessment of 1910 was a little more than %, but City Railway complications raised this ratio to more than % in 1911.

Early in May, 1911, the Supreme Court of the State decreed that he St. Paul City Railway Company must at once comply with the Ordinance of October, 1909, requiring this company to sprinkle its racks. As an immediate alternative, the company agreed with the Commissioner of Public Works to pay the Sprinkling Department for doing the work. The cost of sprinkling streets traversed by the City Railway was to be prorated as to time and space from the totals of the general assessment for the same streets. Form 190 (Fig. 7) was provided for this record.

Form 190 9-19-1911 1M.

Cost of St.Paul City Railway Sprinkling by the City of St.Paul, Minn.

SEASON OF 1911

Street State From Louisa To Concord

City Sprinkling, Begun Mar 31 Finished Nov 15 No.of Months 7.5

For City Ry.Co. Begun May 8 Finished Nov 15 No.of Months 6.2

Width of Roadway 39 Width of City Ry, Strip 19

Time Ratio .83 Space Ratio .486 Assessment Ratio .40

Charge City Railway Co. 22.09

Charge Abutting Property 33.13

Total Assessment 55.22

Standard size 6½ by 4 in.

FIG. 7.

As soon as arrangements could be made, the City Railway Company sprinkled many miles of its track space with road oil.

The occasional itinerary which has been referred to has proved to be a practicable and an excellent feature. It was called for at irregular intervals, and the returns assisted in determining the efficiency of both teamsters and inspectors. Each teamster and inspector is furnished with a specific route sheet, and the care taken to equalize routes is generally appreciated and tends to the good of the service. The routes as traveled are reported on Form 173 (Fig. 8). They are platted on light blue prints. A summary is made of the number of

tanks of water used and of the feet of effective and idle travel in a half day. The 45 sq. miles of sprinkling area are covered by 34 tracings (8½ by 13½ in.) on a scale of 600 ft. to 1 in., and from these tracings the blue prints referred to are made as required.

Among the many owners of the 50 000 lots, more or less, assessed, there are many complainants who desire to know the particulars, and, for the benefit of this class, a special form has been prepared. Occasionally, a correction is necessary; there were only twenty on the first assessment made under this system.

Form 173 3-29-1910 1M.

SPRINKLING DEPARTMENT, ST. PAUL, MINN.

Report on Team Route No. 1 *District No.* 6 July 28 1910

Tank 72 Kranz

QUARTER TANKS 1	2	3	4	Minutes	Side	STREET	FROM	TO	SPRAY O	SPRAY C	O'CLOCK A.M.	O'CLOCK P.M.
			X	5		Rice and Rondo						100
						Rice	Rondo	Sylehart		X		
						Sylehart	Rice	Louis	X			110
						Louis	Sylehart	Rondo		X		
			X	5		Rondo and Louis S.P.						
						Louis	Rondo	Carroll		X		
						Louis	Carroll	Nelson	X			
			X	5		Nelson and Farrington S.P.						
						Farrington	Nelson	Selby		X		

Inspector

Standard size 8¼ by 6⅞ in.

FIG. 8.

The assessment for each ward is cross-checked and balanced to a cent. A concluding word may be said as to the daily reports, Form 150 (Fig. 2). Each section of street has its trip number. The small figures under the trip numbers show the number of trips actually made for the day. Under Route 4 it will be noted that Trip No. 31 lacks the sub-number. The explanation immediately follows. To write "31" is equivalent to writing "Indiana Avenue, from South Robert Street to Hyde Street." On May 18th Trip No. 31 was divided. A portion of it (No. 84) being transferred from Route 4 to Route 7.

Each inspector keeps a consecutive list of last numbers in his district, and can thus assign a new number when necessary.

DISCUSSION

S. WHINERY, M. AM. SOC. C. E. (by letter).—This paper is so interesting and valuable that the reader reaches its end hungry for more information. Mr. Whinery.

It is so seldom that this branch of municipal work receives, from city engineers and city officials, anything like the attention it deserves or that is given to other departments of city work of no greater importance, that any one who takes an interest in the matter must welcome this account of an intelligent and efficient organization for street sprinkling in St. Paul. It is sincerely to be hoped that the author will favor the Society with another paper dealing with experiences and results. It would be very interesting to know about the practical working of the system, the efficiency attained, as compared with the usual unsatisfactory organization—or lack of organization—and the degree in which the service is successful in abating the dust nuisance and in meeting the reasonable demands of the public.

Particularly would engineers be glad to know the detailed cost of the service, reduced to units readily comparable with results in other cities.

The writer would suggest, as the most simple and satisfactory unit of quantity, 1 000 sq. yd. of street sprinkled once, and that all other elements of cost and service be based on this unit. The units commonly used (where statistics are reported at all) are often so indefinite or general as to be of little use for comparison. Thus, the number of miles of street sprinkled through the season is of little value unless one knows the widths of the streets and the number of times they are sprinkled daily, or rather the number of times they are sprinkled during the season.

The method described for assessing the cost of the service on abutting property owners seems to indicate that the width of the street is not taken into consideration. It seems proper that this should be done. Those owning property on a wide street certainly receive more service than those on a narrow street; and as property on a wide street is usually more valuable than that on a narrow street, the owners might justly be required to pay in the ratio of the work done, that is, the area of the street. To provide for this, of course, would introduce another factor into the computation of the assessments, but if the widths (or half widths) of the streets are known, the actual work of computation would not be increased greatly, though the unit on which the assessment is based would be changed from front feet to square feet, or square yards.

Mr. Whinery.

It is not stated how street intersections are dealt with in assessing the cost, though the inference that the cost of sprinkling intersections is taxed on the property owners seems to be warranted; nor is it stated how corner lots, sprinkled on two sides, are assessed.

As it is stated that street oiling is used to some extent, it would be interesting to know the relative cost, efficiency, and general merits of oiling and sprinkling in St. Paul.

It is not only in St. Paul that stand-pipes for supplying sprinkling wagons are regarded as nuisances. Their unsightliness might be overcome in most locations by using a valve and connection placed under the edge of the sidewalk and covered by a hinged plate. The chief source of dissatisfaction is usually the "sloppiness" around these stand-pipes. This is chargeable largely to the carelessness of drivers in allowing the tanks to overflow, and, where the hose is permanently connected to the tank, in allowing the water contained in it to waste on the street after it is disconnected. In this form of connection the trouble would be largely overcome if a valve were placed at the rear end of the hose, and closed before the hose is disconnected from the hydrant. Certainly this trouble can be overcome by the use of appropriate devices and reasonable care on the part of the driver.

While in most cities the municipality supplies, free of charge, the water used for sprinkling, and its value is not charged to the account, it is very desirable that the approximate quantity and cost should be reported, in order that a complete statement of cost per unit area may be deduced.

Mr. Blanchard.

A. H. Blanchard, M. Am. Soc. C. E.—This paper brings to the attention of municipal engineers the value of cost data and accurate records of construction details. The speaker, however, only refers to this feature of the paper in order to express his appreciation of the author's services in bringing before the Society a comprehensive plan covering cost data and records in a field in which very little work of this character has been done.

A paper on the broad problems of the economics of street watering seems to be opportune. Up to this time there has been comparatively little discussion relative to the efficiency of street watering. In considering the problem from this standpoint, a question which occurs to any one interested in the subject is: What are the fundamental reasons for sprinkling streets with water? There are engineers who claim that the reason is mainly to lay the dust. Street sprinkling, as done by these engineers, is used for that purpose not only on macadam roads, but on pavements of all types. If pavements are properly cleaned (by methods which are adaptable in practically all American cities), is there any necessity to sprinkle them to lay the dust? The answer is in the negative, because, with a proper system of street cleaning, dust should not exist in such quantities as to require any

process of dust laying. The question of laying dust on macadam roads naturally brings up for consideration the relative economics by watering and by other methods which have come into use within the past decade. This feature will not be considered by the speaker, as it has formed the subject of many discussions before this Society.* Mr. Blanchard.

Another reason advanced by some municipal engineers is that street watering materially cools the atmosphere. These engineers, however, use street sprinkling methods from the middle of April to the middle of October. It is doubtful if, in the majority of cities, engineers could state conscientiously that it is necessary to cool the atmosphere during two-thirds of the time when streets are watered. It would be of material value if a series of experiments should be undertaken to determine how much the atmosphere is cooled within a certain reasonable distance above the pavement and on adjoining property by street sprinkling with water. G. A. Soper, M. Am. Soc. C. E., who has spent considerable time in investigating the subject, is particularly positive in the statement that no cooling effect is appreciable. Although it is admitted by many that it may not be appreciable from the standpoint of the actual temperature, it is believed, however, that on macadam roads, under certain local conditions, there is a cooling effect—at least mentally. One excellent example of this effect is seen in the method used in Monte Carlo. Although the macadam roads of the Principality of Monaco are rendered dustless in the main by the use of bituminous materials, in the instance of the light-colored limestone macadam roads in the vicinity of the Casino, surrounding the beautiful gardens laid out in front of that magnificent yellow limestone building, the streets are watered not only to give the effect of coolness, but also, from the esthetic standpoint, in order that the limestone macadam thus treated will harmonize with the beautiful buildings and residences which border these boulevards.

A third reason which has been advanced by some engineers is that street watering serves to clean the streets. The speaker does not believe that it is a question for consideration whether street sprinkling properly conducted does or does not clean the streets. Certainly, it is usually impracticable to clean macadam roads by watering without the creation of mud or the disintegration of the surface. With the figures at hand, it certainly is not economical to clean pavements by street sprinkling when they are used under conditions for which they are suitable and economical.

C. L. Annan, M. Am. Soc. C. E. (by letter).—In reply to Mr. Whinery, the following statement of unit costs is presented: Mr. Annan.

Total travel of 80 teams per day.....	5 248 000	lin. ft.
Total effective travel (76%).....	3 883 520	" "
Total area (width of spray, 22 ft.)....	9 493 050	sq. yd.

* *Transactions*, Am. Soc. C. E., Vol. LXV, pages 462 to 466, and Vol. LXXIII, pages 33 to 43.

Mr. Annan.

Daily expense:

80 teams @ $100 per month	$267
10 inspectors @ $75 per month	25
Stand-pipe repairs, average	15
Sprinkler " "	13
2 000 000 gal. water @ $90	180
Total	$500

Consequent cost of sprinkling 1 sq. yd. once.	0.00527	cent.
Average cost of sprinkling 1 sq. yd. four times per day for 210 days	4.4	cents.

During the summer of 1910, 80 000 gal. of a 45% liquid asphalt, heated to 150° Fahr., were applied to 300 000 sq. yd. of macadam surface, the cost per square yard being as follows:

Cleaning street surface	0.35	cent.
Road oil	1.00	"
Application	0.15	"
Total	1.5	cents.

Three applications would be required for the full season, which would make the cost practically the same as for water sprinkling.

There was much opposition and complaint of damage and annoyance by residents along the smeared streets, however, and, furthermore, as the oil dried out, it left the street surface in a badly disintegrated condition. During the following year a 20% oil was used. No heating was required; and three applications were sufficient for the season. The surface damage was less, but the popular outcry was not much abated.

In making the assessment, the factor of street widths is provided for in the trip element. Three trips are necessary to cover some wide streets; one trip suffices for a narrow street. The same frontage is assessed three times as much in one case as in the other, as a trip is a single line of travel in one direction.

The total cost of the season's sprinkling is prorated to the frontage participating. The law provides that 10% of this cost may be for equipment. The sprinkling of street intersections must also be regarded as a general charge, as there is no provision for payment otherwise. The law has also been construed to prohibit rebates for corner lots abutting on two sprinkled streets.

The matter of the concealed hydrant received attention early in 1910. Neither of the two eastern manufacturers approached could furnish just what was wanted, and it was late in the season before the desired article was obtained. In the meantime, the conclusion had been reached that the main objection to the stand-pipe is not its un-

sightly appearance. The ungainly sprinkler itself is no more esthetic, but worst of all is the faithful but inconsiderate horse at his 5 to 10-min. stands. Mr. Annan.

The teamster would view the innovation with no favorable eye. He would consider it a hardship to be forced to descend and climb again whenever he filled his tank, to say nothing of lifting the hydrant cover off and putting it on, and of stowing his loaded hose. The new hydrant was never put in.

The mud at the base of the stand-pipe, of course, must be eliminated as soon as possible by draining and paving, to insure the departure of a free tank.

When the St. Paul pavements are bare in freezing weather they are sprayed with calcium chloride (1 lb. per gal.) dissolved with a steam jet.

The following figures are approximately correct for a period of 3 weeks in December, 1910:

27 tons calcium chloride @ $14	$378
8 tons coal @ $4.75	38
4 teams, 18 days, @ $4	288
3 men, 18 days, @ $2	108
Total	$812

At intervals during the period, 100 000 sq. yd. of pavement were treated.

Mr. Blanchard asks: "If pavements are properly cleaned, * * * is there any necessity to sprinkle them to lay the dust? The answer is in the negative * * *."

Another answer is that the use of methods commonly considered effective does not down the demon "dust." Until perfection of method can be more nearly reached, a little water must be used here and elsewhere. Hand sweeping by day and power flushing by night and day do not hold dust in subjection. It is unceasingly making. It is tracked and blown in from adjoining unpaved streets, and arises from the sites of building operations. Whatever the theory, the condition must be met.

A light oil, especially designed for use on pavements, was tested last year on asphalt, brick, and stone surfaces in St. Paul, but with unsatisfactory results.

The vacuum cleaner may eventually do away with the use of the universal solvent, but it will be necessary to apply it to the sidewalks as well as the roadways, and even then will it get the impalpable, harassing dust emanating from all sources of friction in the busy life of a city's streets?

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1242

CONSTRUCTION OF A HIGH-SERVICE RESERVOIR AT BALTIMORE, MD.*

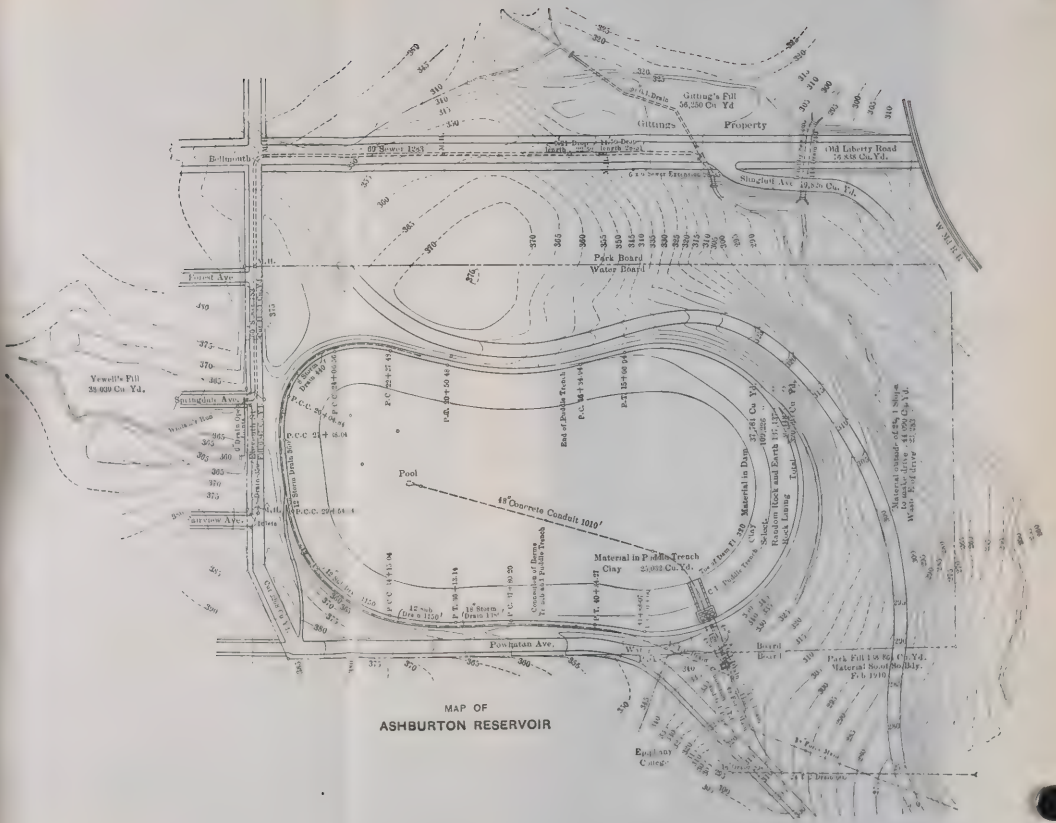
By P. A. Beatty, M. Am. Soc. C. E.

The new high-service reservoir, Lake Ashburton, constructed by the Water Department of Baltimore in 1908-10, is on a site near the intersection of Slingluff Avenue and Liberty Road, and only a short distance west of the Western Maryland Railroad. The spot selected is between two low spurs which run eastward from a plateau, about 450 ft. above mean tide, and terminate in the low ground at Pecks Run. Between these spurs there flowed a small stream, Williams Run, carrying the cesspool overflow from a small suburban development and the storm-water flow from a few hundred acres.

Sub-surface investigations indicated that, underlying a foot of top soil, there was a stratum of clay, from 1 to 4 ft. in thickness, beneath which the formation consisted of "rotten rock" (a decomposed serpentine) of varying depth, blanketing and lying between ledges of semi-decomposed rock and hard serpentine, more or less shattered, and with occasional nests of boulders. A portion of the site which was in timber was cleared and grubbed under a separate contract.

The work to be done consisted of the construction of an earthen dam, and the excavation of a basin, with appurtenant structures; and also the construction of a concrete storm-water sewer for the diversion of Williams Run.

* Presented at the meeting of October 16th, 1912.



MAP OF
ASHBURTON RESERVOIR

The work was let on a unit price basis, the reservoir work to Lane Brothers Company and Jones, of Altavista, Va., and Baltimore, the time being limited to 400 working days; and the diversion sewer to C. B. Clark and Company, a local firm, the limit being 200 working days. In the ten bids received for the construction of the reservoir, the total prices ranged from $494 810 to $837 241, and the working time from 200 to 700 days. In the thirteen bids received for the construction of the diversion sewer, the total prices ranged from $33 572 to $73 986, and the working time from 100 to 650 days.

Except for some topographical modifications, the work was completed substantially as laid out originally. Between the Western Maryland Railroad and Ninth Street the grade of Liberty Road was raised, and Slingluff Avenue was brought to the grade of Liberty Road on a curved fill with a 4% grade. This necessitated the construction of a 6 by 6-ft. box culvert, the extension of the diversion sewer 98.3 ft., caring for the flow from an existing box culvert, and the construction of a 9½-ft. arch culvert 240 ft. long to carry Pecks Run. The latter was built by the Baltimore Sewerage Commission. These modifications are shown on Plate VII, which covers the work as completed.

The original plans provided for a dam to be built in 6-in. lifts, thoroughly rolled, and for a reinforced concrete diaphragm against the outer wall of the cut-off trench, the trench to be back-filled with rolled or rammed clay. The former was changed to a fill deposited under water; the diaphragm in the cut-off trench was eliminated, and the trench was back-filled with clay, deposited under water.

Ground was broken on July 6th, 1908. The soil within the basin area was removed with wheeled scrapers and stored along the margins of the Water Board's property where convenient, after which the clay was similarly removed and stored near the soil banks, or within the basin area, adjacent to the cut-off trench. Meanwhile, that portion of the dam site which lay in the valley was being cleared of boulders and stripped of loam, roots, muck, etc., this waste material being deposited beyond the toe of the dam proper, and in the base of the East Drive fill. Several small springs, discovered near the bottom of the valley in the dam foundation, were led beyond the toe in blind drains.

Williams Run was diverted at the point where it crossed Contour 320, and was carried along the side of the south spur, crossing the

dam site at Elevation 315, being conducted over the cut-off trench in a 4 by 4-ft. box.

The cut-off trench was excavated with plows and scrapers, and with carts on the hillsides where it could be benched, derricks being set up to cover the work as the excavation deepened. The material through which the trench was sunk consisted of rotten rock, semi-decomposed rock, hard seamy rock, and hard serpentine ledges. At one point there was a mass of boulders, many of them containing 1 cu. yd. or more, extending from the surface to bed-rock and bedded in a matrix of gray, fully decomposed rock. On the north side, the trench cut a dike of felsite, which crossed it nearly at right angles, was about 18 ft. thick between nearly vertical faces, and extended from the surface to the bottom of the trench, about 62 ft. deep at this point.

When bed-rock was reached, it was found to be more or less seamy in places, making it necessary to continue the excavation until the conditions became satisfactory. Very little water was encountered at any point, the few small springs exposed being led to 2-in. stand-pipes through smaller pipe connections, and these stand-pipes were ultimately grouted. The average depth of the trench was 40 ft., the length 2 100 ft. The timbering consisted of 2 by 10-in. sheeting, 6 by 8-in. rangers 4 ft. 6 in. from center to center, and 6 by 8-in. braces 8 ft. apart.

In August a shovel was placed in Eleventh Street, and the fill at Williams Run was made. The water was allowed to pool to a depth of about 10 ft. on the upper side, and was carried thence through two lines of 18-in. pipe through the south end of the fill. Later, the pool was filled from the Eleventh Street cutting. When the run was finally diverted through the sewer, the pipes were followed into the fill and removed as far as practicable; they were then plugged with concrete, and a mass of clay was tamped into the excavation. An excavation was made afterward in the fill to the west of Eleventh Street, and a sub-drain was laid in a shallow trench in the original soil, well surrounded with broken rock, the purpose being to prevent the collection of a pool of stagnant water in the fill and the attendant danger of pollution of the water in the reservoir.

Eleventh Street being completed, the shovel was removed to the basin and began work on the north side, following down the slope with a series of curved cuts until Elevation 355 was reached, where a 15-ft.

94A

Liberty Road
Park Board Property
Camp of Contractors
Forest Ave.
Springdale Ave.
Powhatan Avenue
New High Service Reservoir
Exc. Remaining in Basin
15 670 Cu. Yds.
Dikes
Puddle Trench
Puddle Wall
Outlet Pipe
Office
Dwellings
Mess Hall
Blacksmith Shop
Water-Tower
Stables
Coal Vault
Pump House
Engine House
Boiler House
Chimney
Tank House
Filter Gallery
Filter Beds
Clear Water Basin
Wash Water Tank
Gate House
Meadow
Stream
Sligo Creek
East Driveway
Exc. 1139 Cu. Yd.
Park Board Property
Waste 138 Sol. Cu. Yd.
Placed
Embankment
Reservoir
Embankment
Reservoir
Embankment
Reservoir

berm was left; thence, the shovel followed the slope and shape of the reservoir to the bottom, at Elevation 320, after which successive parallel cuts were made until the cutting of Shovel No. 2 was joined.

Before the dam site was prepared for rolling, this shovel stored select material on the hillside immediately east of the cut-off trench, the random rock and earth being deposited in the base of the driveway fill, and tracks being laid across the valley on an embankment which had been built with carts.

On the north side, the first layout of tracks from the shovel ran eastward on a descending grade on the hillside, crossing the cut-off trench on a bridge, and curved northward through a shallow cut in the end of the spur to a switchback near the intersection of the north boundary line with Contour 315, from which the cart fill just mentioned was reached by a sharply falling grade. As this fill increased in height, the grade of the switchback became flatter and ultimately was reversed, giving a steep grade from the switchback to the dam. It was then abandoned, and the tracks were laid as shown on Plate VIII.

As the shovel descended the slope, switchbacks were necessary to get the spoil out of the basin. Plate VIII shows the loading track running to a switchback in the eastern end of the basin, from which the running track ascends the slope westward to a second switchback at the west end, at Elevation 350; thence it ran eastward, crossing the berm and curving northward on a descending grade to Liberty Road. Near the water tank, Track "C" curved around the inner slope of the dam. A second track, "A," gave access to the outer slope.

Plate VIII shows the Liberty Road and Gittings fills completed, and the Slingluff Avenue fill nearly finished, with the dam brought to Elevation 345.

Shovel No. 2 was started at Contour 320 on the south side, having first made a cut through the south spur immediately east of the gate-house site, through which a running track was laid. In the first position, with the shovel and loading track on the reservoir floor, the track reached the cut east of the gate-house by an ascending grade, from which point, with a descending grade, it curved around the end of the spur and reached the dam through the East Drive cut.

A switchback, near the south boundary line, gave access to the fill on the Park Board's property south of the gate-house. With this

location of the tracks, the dam was brought to Elevation 300, and the first lift of the fill on the Park Board land was made. Two relocations of tracks were necessary, the last being that shown on Plate VIII, from which position the dam and the last lift of the Park fill were completed, and the fill for Powhatan Drive was made. The last relocation was made when Shovel No. 2 was moved from the floor of the basin to the top and began to work down the slope, which was rendered necessary, in part, by the scarcity of proper material for construction in the lower cut.

Plate VIII shows that Shovel No. 2, having worked down the slope, is again on the reservoir floor, removing the long, narrow ridge between the foot of the slope and the previous cutting. The track runs from the shovel to a switchback near the west end of the basin at Elevation 340, thence eastward along the slope to a point near the gate-house, where it bifurcates, one branch being continuous with Track "A" from the north side, the other, "B," lying between "A" and "C." "A" is in position for raising an outer embankment of random rock and earth; "C" can place either "rock lining" or clay; while "B" is just completing a lift of select material. The fill on the Park land is shown as completed, the terraces indicated by the contours being dressed to a regular slope later.

The necessity of getting out certain classes of material as they were needed required considerable moving of the shovels back and forth, but less, on the whole, than might have been expected, as it was usually possible, by careful planning, to have the shovels working in the materials needed, and thus very little time was lost.

The total quantity removed from the basin below the clay was 582 458 cu. yd., of which 161 307 cu. yd. were earth, 98 752 cu. yd. were loose rock, and 322 399 cu. yd. were solid rock. The only unusual features connected with the construction were the methods used on the dam and the cut-off wall. A short description of the excavation of the trench for the cut-off wall has been given. When the question of the elimination of the reinforced concrete diaphragm was taken up, a series of experiments was undertaken with rammed clay and clay deposited in water. Clay and rotten rock, rammed in cylinders and placed under water pressure, were tested for penetration. When compressed in cylinders, under a pressure of 150 lb. per sq. in., the clay weighed 131.6 lb. and the rotten rock 129 lb. per cu. ft.

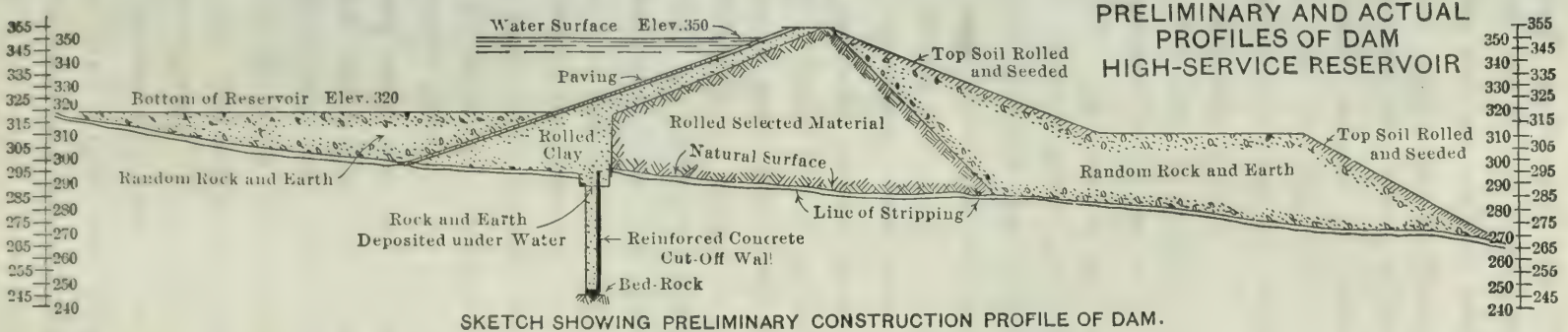
FIG. 1.—CUT-OFF TRENCH AT INNER TOE OF DAM, LOOKING NORTH.

FIG. 2.—LOOKING NORTH OVER RANDOM ROCK SECTION OF DAM FOUNDATION. TO LEFT, EMBANKMENT AT TOE OF SELECT SECTION. TO RIGHT, EMBANKMENT ON EAST DRIVEWAY FILL. DEPRESSION TO BE FLOODED AND FILLED WITH RANDOM ROCK AND EARTH.

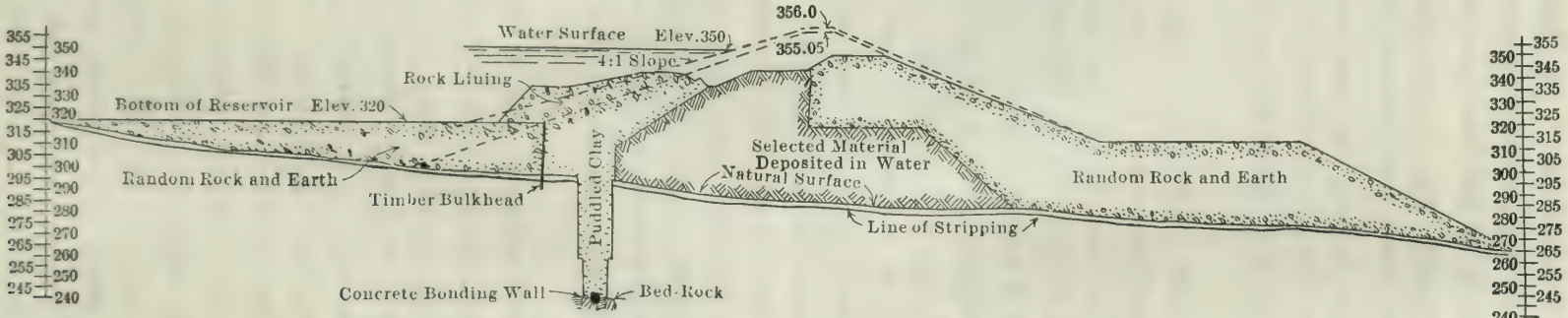
Several 9 by 12-in. boxes, 8 ft. long, constructed so that the sides could be removed by loosening nuts and rods, were placed on end and filled with water. Clay was shoveled into some of these, and rotten rock into others, and some were filled with mixtures of the two in varying proportions. In several of the boxes a number of clods and lumps of clay and rotten rock were placed. About two weeks later, the sides of the boxes were removed and the contents examined, and found to be compact and homogeneous, the clods having fused with the surrounding material and practically disappeared. The material was compact and tough, and only slightly damp. The clay weighed 125 lb. and the rotten rock 118 lb. per cu. ft. This material had been subjected to no extraneous pressure; in the dam, the weight would certainly be very much greater, owing to the pressure exerted by the superincumbent material.

A section of the trench having been completed, several feet of clay were rammed into it in 3-in. layers, a picked foreman having charge of this work. It presented a good appearance, and seemed to be quite solid. Several 20-ft. lengths of 2-in. pipe were then set vertically at various points, their lower ends being deeply sunk in the clay (which was firmly rammed around them). Then these pipes were filled with water. The water usually forced its way to the surface around the pipes, but in several cases found lines of weakness in the clay and appeared several feet away. Several pits were also sunk in the clay and filled with water; two or three of these, placed directly beneath the braces, leaked, showing that the ramming was imperfect at these points. This led to the conclusion that satisfactory results could not be obtained by ramming. The cost of this work was 87.4 cents per cu. yd. It was then decided to fill the trench with clay deposited in water, using a certain percentage of rotten rock where practicable, care being taken to insure thorough mixing.

It was impracticable to remove the sheeting, but every fifth or sixth plank was cut away as the clay arose, permitting the puddle to fill the spaces between the planking and the trench walls. Heavy vertical timbers were placed at short intervals and braced hard against the rangers. The lower rangers were held by the cantilever ends of the verticals, and the original bracing was removed as the clay reached it. Usually, two sets of cross-braces were removed before the verticals were shifted. When the clay neared the third set, the jack-braces



SKETCH SHOWING PRELIMINARY CONSTRUCTION PROFILE OF DAM.



SKETCH SHOWING ADOPTED CONSTRUCTION PROFILE OF DAM.

FIG. 3.

FIG. 4.—LOOKING EASTWARD FROM NORTHWEST CORNER. TERRACED CUTTING BY NORTH SHOVEL.

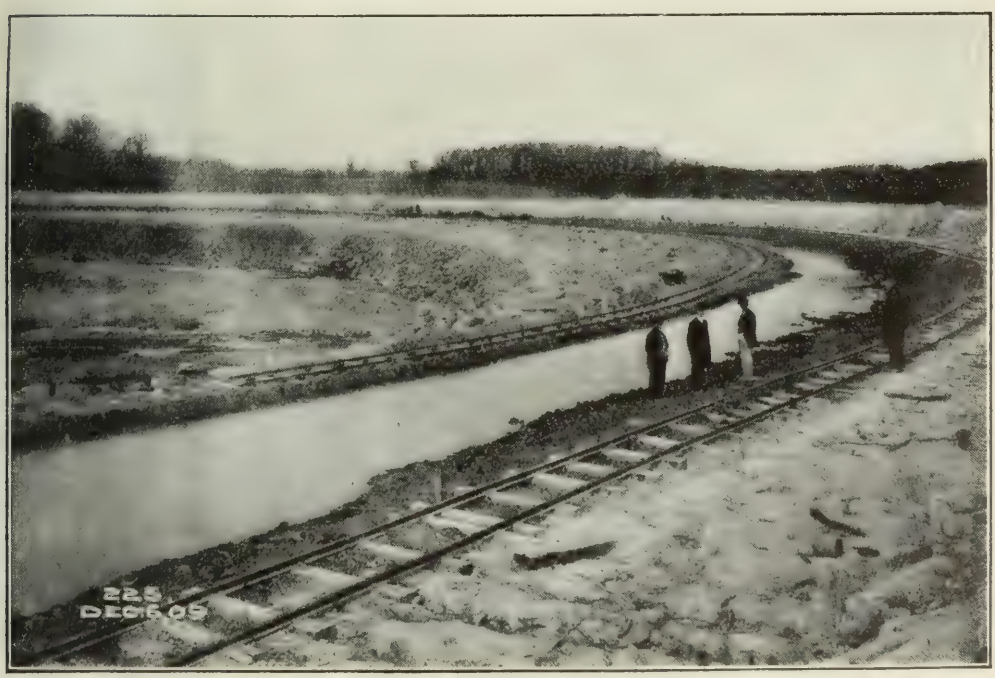


FIG. 5.—LINING WITH CLAY. TRACK ON COMPLETED LIFT OF SELECT SECTION. BACKGROUND, NEXT LIFT OF RANDOM ROCK. TO LEFT, RESERVOIR FLOOR, ELEVATION 320.

FIG. 6.—DRESSING INNER SLOPE OF DAM. THE ADDITIONAL MATERIAL REINFORCING THE ROCK LINING AGAINST SLIPPING CAN BE SEEN DISTINCTLY.

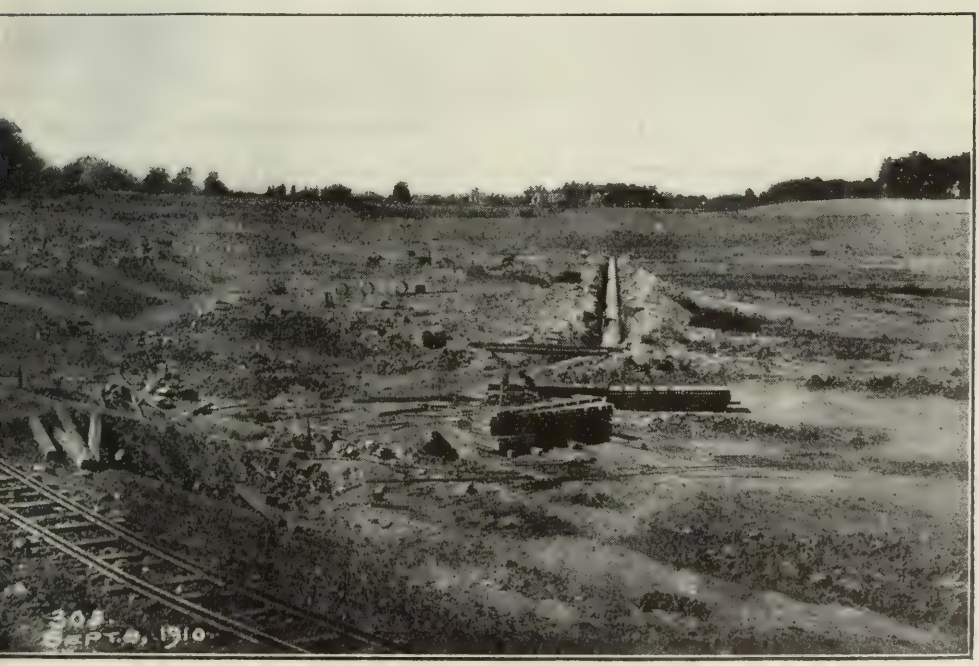


FIG. 7.—LOOKING WESTWARD FROM DAM. LOCK-JOINT INFLUENT PIPE IN CENTER.

were eased off and the verticals were lifted and rebraced, auxiliary braces being placed where required during this change.

Through the deeper part of the trench a concrete toe-wall had been placed. This contained the drainage pipes from the springs, leading to the stand-pipes. This wall was not carried the entire length of the trench, the irregularity of the bottom seeming to form a sufficient obstruction to the passage of the water.

In placing the clay, picked foremen were employed, and they were constantly supervised by the inspectors. Care was taken to remove all foreign matter, and lumps were broken to the size of an orange, all stones of greater size being rejected. The shovelers were directed to cast the clay and rotten rock so that it would scatter in falling.

When rotten rock was accessible, from 40 to 50% was used. It was cast in from one side while the clay was being cast in from the other. The desired proportions were obtained by regulating the number of shovelers on each class of material.

The water was usually about 4 ft. deep at the beginning of each lift, and the casting was stopped when the clay was within about 1 ft. of the surface of the water. Fine roots, chips, cinders, etc., not noticeable in the clay, floated to the surface, and were kept skimmed off.

As the puddle reached the ground surface in the lowest part of the valley, clay bulkheads, several feet in height, were built across the trench, water was turned in, and a new lift was begun. In this way the puddle was carried up the hillsides in successive terraces, filling the trench to the surface throughout. Very little trouble was experienced from the slipping of these dry clay bulkheads, although founded on freshly puddled clay; there was usually some movement, accompanied by a slight heaving of the clay below the bulkhead, but it was so small that it was of no consequence. The city paid 25 cents per cu. yd. for the clay thus deposited, which covered all charges from the storage pile to its place in the trench, and the placing and removal of the timbering rendered necessary by this method. The clay settled about 2 ft. in two or three months, but little settlement took place thereafter. Investigations made several months later showed that the clay was compact, firm, and slightly moist at a depth of 7 ft. below the surface.

The first fill across the valley was at the toe of the East Drive,

and was made with carts; on this a track was laid, connecting with the switchback on the north side, and later joining the track from Shovel No. 2, passing through the cut on the East Drive south of the dam.

A second embankment was built across the valley at the line of junction of the select and random rock sections, and Williams Run was turned into the depression thus caused, forming a pool, into which the random rock and earth were dumped.

There was not enough earth in the filling material to make an impervious embankment, nor was this desired, the intention being to obtain a compact but somewhat pervious fill, which would allow the escape of any water percolating through the select section of the dam. This method was followed on the random rock section until the narrowing of the dam, as the height increased, rendered it impracticable.

The question of building the entire dam by depositing the materials in water had been under discussion for some time, and it was finally decided to adopt this method, the contractors making a satisfactory rebate on the cost of this portion of the work. It was expected that the principal difficulties would be the placing of the materials under water in their designated positions and the preservation of the proper slope on the inner side.

The following method was adopted: The inner slope of the random rock section was given a lining of select material dumped from cars, to prevent the escape of water from the proposed pool; and a narrow embankment was built across the valley just outside of the cut-off trench, to prevent the water from entering the latter.

This basin was then flooded, and select material was dumped along the outer edge of the pool, the track being thrown inward as the fill widened. The tracks had to be kept 2 ft. or more above the surface of the water, the fill showing a tendency to slip out, particularly where the water had considerable depth, and the submerged portion took a very flat slope, the finer portions of the select material running far ahead and forming a bed of mud, in which the solid materials were thoroughly bedded.

This tendency of the toe to advance had to be watched carefully, in order to prevent it from encroaching on the clay section, a difficulty which was overcome later by raising a low ridge of clay on the line between the select and clay sections. A surprising quantity of air

FIG. 8.—SWALE AND BERM AT ELEVATION 355. DAM IN BACKGROUND.

FIG. 9.—LOOKING SOUTHWARD ACROSS BASIN. DAM AND GATE-HOUSE IN BACKGROUND.

seemed to be entrained in the select material, and somewhat less in the clay. That this was expelled at an early date seems conclusive, in view of the small amount of settlement in the finished structure.

The clay having reached the top of the cut-off trench, a wooden bulkhead was erected, 12 ft. from its inner edge, on the line of the proposed extension of the clay above the ground surface and the fill inside of the dam, which was being placed to bring the floor of the reservoir to Elevation 320.

A trench was excavated to a depth of several feet, in which a continuous sill piece was laid, to which posts, 6 ft. from center to center, were fastened with ½-in. iron pins passing horizontally through both posts and stringers. The posts were braced in a vertical position, and the trench was packed with rock. To these posts 2-in. planks were spiked as the fill was raised.

The first lift of the select section having reached the clay line, clay was brought from the storage banks, being loaded on the trains with scrapers by means of a trap. This clay was dumped on the edge of the select embankment and cast into the water, the same care being taken as in filling the cut-off trench, the limit of the clay puddle being the bulkhead.

Meanwhile, the random rock and earth section had been started on the next lift and lined as before to form the outer embankment of the next pool, the depression was again flooded, and the foregoing operations were repeated. There was considerable difficulty in keeping the bulkhead standing, but it was accomplished by judicious dumping on the inside to counterbalance the pressure of the clay. These operations were repeated on each lift.

Above Elevation 320 it became necessary to raise an embankment on the inner as well as on the outer side, on a 3 : 1 slope, to conform to the profile. For this purpose, the best rock that the shovels could reach was used to form the "rock lining," the first lift being deposited at the intersection of the 3 : 1 slope with the floor at Elevation 320, and reaching from hill to hill. This embankment was 5 ft. high, and just sufficiently broad on top to hold a track. A light blanket of clay was dumped on the pool side and a new pool was formed. Water was delivered to the dam site by a 6-in. main connected with the city supply. From this elevation up the pools were usually 3 or 4 ft. deep, the lifts being about 5 ft. Each successive lift of rock lining over-

lapped the clay sufficiently to give the required slope, and this operation was repeated until the top of the dam was reached.

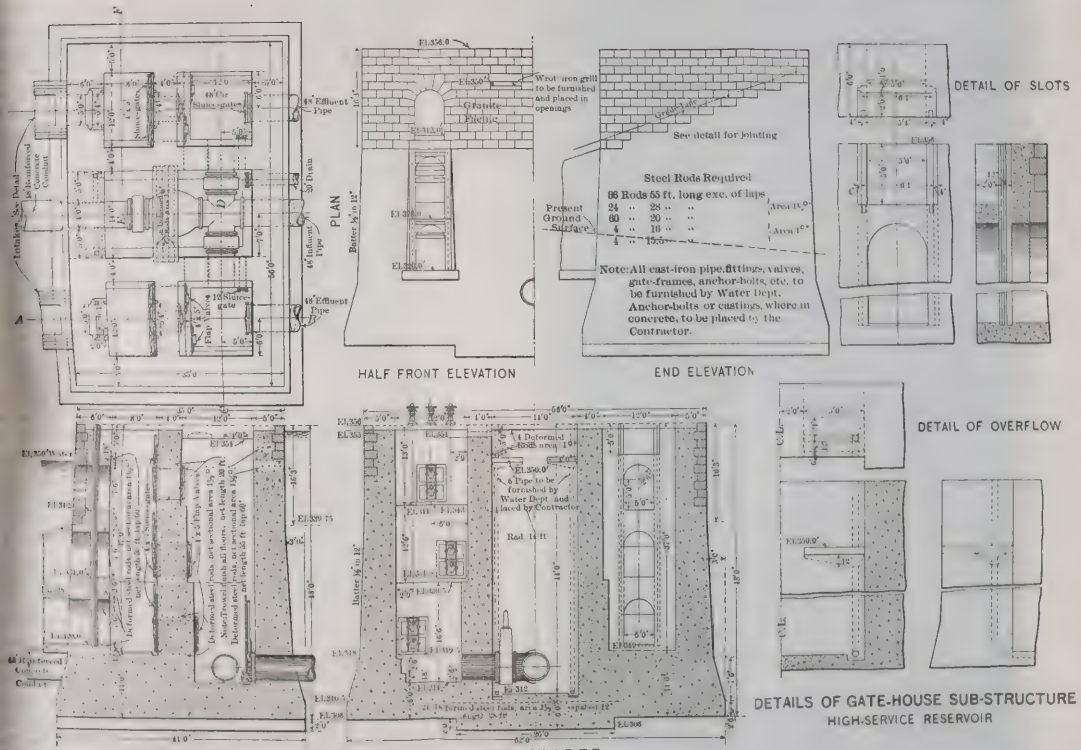
In order to insure a tight job at the gate-house, the space between the walls of the foundation pit and the concrete was filled with clay puddle, and a mass of clay, from 8 to 10 ft. in thickness, was placed entirely around the structure, extending from the foundation to above the water line. This was brought up with the successive lifts of the dam.

Everything progressed in a satisfactory manner until an elevation of about 335 was reached, when, a little north of the middle of the dam, a movement was observed in the rock lining. At the time, rock lining was being placed, and a considerable quantity had been dumped above the place in question. The movement consisted of a steepening of the slope a short distance above the toe, covering a space of 200 ft. or more, and a swelling of the surface. As soon as this came to the writer's notice, the dumping was stopped at this point, and the track was thrown down to the part affected. Rock was first dumped over the toe to buttress it and prevent kicking out, after which the track was thrown up the slope, and a good weight placed on the swelling. By midnight, it was thought that the danger was over, and the men quit work. In the morning, as early as possible, the Water Engineer and his assistant made a thorough examination, and found that for about 500 ft. the toe of the rock lining, Elevation 320, had moved inward from 1 to 3 in., the slope appearing to be unaffected except at the point mentioned.

It was decided to buttress the toe from side to side of the valley, and weight the slope with a gradually diminishing mass of rock, giving it a 4:1 slope between the hills, warping back to a 3:1 slope as the ends were approached. A line of centered stakes was placed across the disturbed portion of the dam and kept under observation for several months, but no additional movement was observed, nor has there been any sign of movement in any portion of the dam since.

In June, 1909, several test-pits were sunk in the select material between the rock section and the pool for the purpose of examining the condition of the material which had been deposited under water, and to get an idea of the quantity of seepage. These pits penetrated two lifts, reaching a third, the bottoms being 7 or 8 ft. below the surface of the pool. No. 1 pit, 12 ft. from the edge of the pool, on

110A.


PLAN
HALF FRONT ELEVATION
END ELEVATION
DETAIL OF SLOTS
DETAIL OF OVERFLOW
SECTION A-B
SECTION C-D E-F
Steel Rods Required
86 Rods 55 ft. long exc. of laps
Present Ground Surface
Note: All cast-iron pipe, fittings, valves, gate-frames, anchor-bolts, etc. to be furnished by Water Dept. Anchor-bolts or castings, where in concrete, to be placed by the Contractor.

DETAILS OF GATE-HOUSE SUB-STRUCTURE
HIGH-SERVICE RESERVOIR

he third day contained 2 ft. of seepage. Nos. 2 and 3, 35 ft. from he pool, remained dry. No. 4, 10 ft. from the pool, on the third day contained 3 ft. of water, and the bottom showed a tendency to heave. Later, the pool was drained, and an examination was made of the bottom, which consisted of gray mud in which matrix all the stones were thoroughly bedded.

The dam was finished at Elevation 356, 1 ft. above the grade called for by the plans, the last 3 ft. being placed dry. The extra foot was added for settlement. It seems, however, that the settlement was about completed before the crest was placed. Stakes were set every 50 ft., and observed for a year, during which period the maximum observed settlement was 0.2 ft. It was impracticable to ascertain the shrinkage during construction, as the entire surface was being worked over, affording little opportunity to maintain a permanent bench.

The total quantities placed in the dam were: 37 784 cu. yd. of clay; 109 226 cu. yd. of select material, and 137 137 cu. yd. of random rock and earth. The East Driveway fill, outside of the dam, contained 67 373 cu. yd. of random material, and the cut-off wall, 25 032 cu. yd. of clay.

At the west end of the reservoir, a cut-off trench was excavated well into the rock, with an average depth of 20 ft., and extending into the hills to the north and south. This was filled with puddled clay, which was brought up as a core-wall to the berm level, random material being placed on either side to support it above the natural ground. The Liberty Road fill was begun from a trestle, erected along the south side of the road, the caps and stringers being removed later. In the track giving access to this fill, 132 800 cu. yd. were placed. A bench was cut along the south slope of the fill, corresponding to the grade of Slingluff Avenue, and 18 000 cu. yd. were used in connecting the two streets. West of Eleventh Street 38 000 cu. yd. were wasted in the Williams Run valley, 159 000 cu. yd. were placed on the Park property south of the gate-house, and 2 800 cu. yd. were placed in the basin to bring the floor to Elevation 320.

To take care of the surface and sub-surface flow which would approach the reservoir from the west end, a trench has been excavated at the outer edge of the berm, and extends from a point near the north end of the dam around to meet the cut-off trench at the south end, with the bottom on a 5% slope. Along the outer side of this

trench there is a 6-in., open-joint sub-drain, surrounded with broken stone, to intercept any seepage which might reach the trench; 3 ft. above this there is laid a line of pipe, connecting with the surface by intakes placed 250 ft. apart, to carry off all surface water flowing toward the reservoir. The trench was back-filled with clay. The two lines of pipe discharge into a manhole west of the gate-house, from which there is a line of pipe connecting with the 20-in. drainage line from the gate-house, running to a manhole on the Park property, whence the flow passes through a 24-in. pipe to a point near the bank of Pecks Run.

Beginning at the north face of the gate-house, the first joint being concreted into the wall, and making connection with the 48-in. force main, a 48-in. reinforced concrete, lock-joint pipe (Meriwether system) extends 1 010 ft. to a pool near the western end of the basin, forming a continuation of the force main. This conduit is laid in the reservoir floor, and crosses the cut-off trench on a reinforced concrete beam. A concrete collar has been placed around the pipe at the cut-off trench.

The force main passes through the central chamber of the gate-house, where there is a check-valve and a four-way branch, and is controlled by a gate-valve. Between the gate-house and the gate-vault, there are three lines of 48-in. pipe. These are connected by a four-way branch in the gate-vault, where there are additional gate-valves. The west line extends through the south wall of the gate-vault, and is closed with a cap. The central line—the force-main—extends to the North Avenue main, which is laid in Seventh Street after leaving the Park property.

From the central chamber of the gate-house, a line of 20-in. cast-iron pipe, for drainage purposes, extends to a point south of the gate-vault, which it also drains through a 2-in. pipe connection; from this point the line is of double-strength vitrified pipe, laid in concrete. Through this pipe and a blow-off at the lowest point on the force-main the reservoir is drained.

The gate-house substructure, extending from Elevation 308 at the foundation to Elevation 356 at the floor level, contains a central and four side chambers. The four-way branch in the central chamber, connects with the south side chambers, and is controlled by gate-valves, operated from the central chamber. The side chambers discharge through the two lines of 48-in. effluent pipe, above mentioned, and

into the central line at the gate-vault, or directly into the force-main in the central chamber, back of the check-valve, and are controlled by sluice-gates.

Three 4 by 5-ft. openings, at Elevations 320, 331, and 342, pierce the walls between the north and south chambers, being controlled on the north and south sides by sluice-gates and flap-valves, respectively. The north chambers are connected with the lake through arched openings at the elevations above given, and have stop-log grooves.

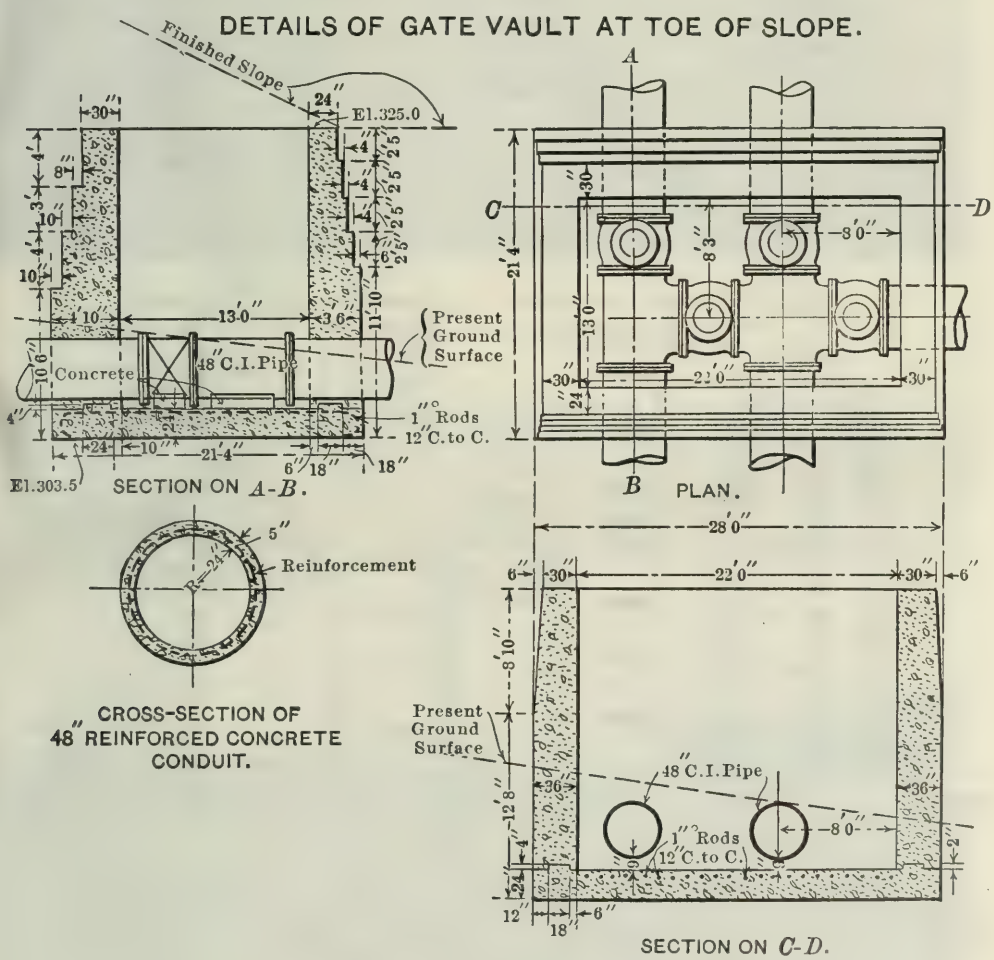


FIG. 10.

Reinforced concrete intakes conduct the water through the dam to the gate-house. At Elevation 350 two overflow openings drain through two lines of 6-in. pipe to gutters on each side of the floor of the central chamber. The central and south side chambers are connected by 12-in. drain pipes, controlled by sluice-gates.

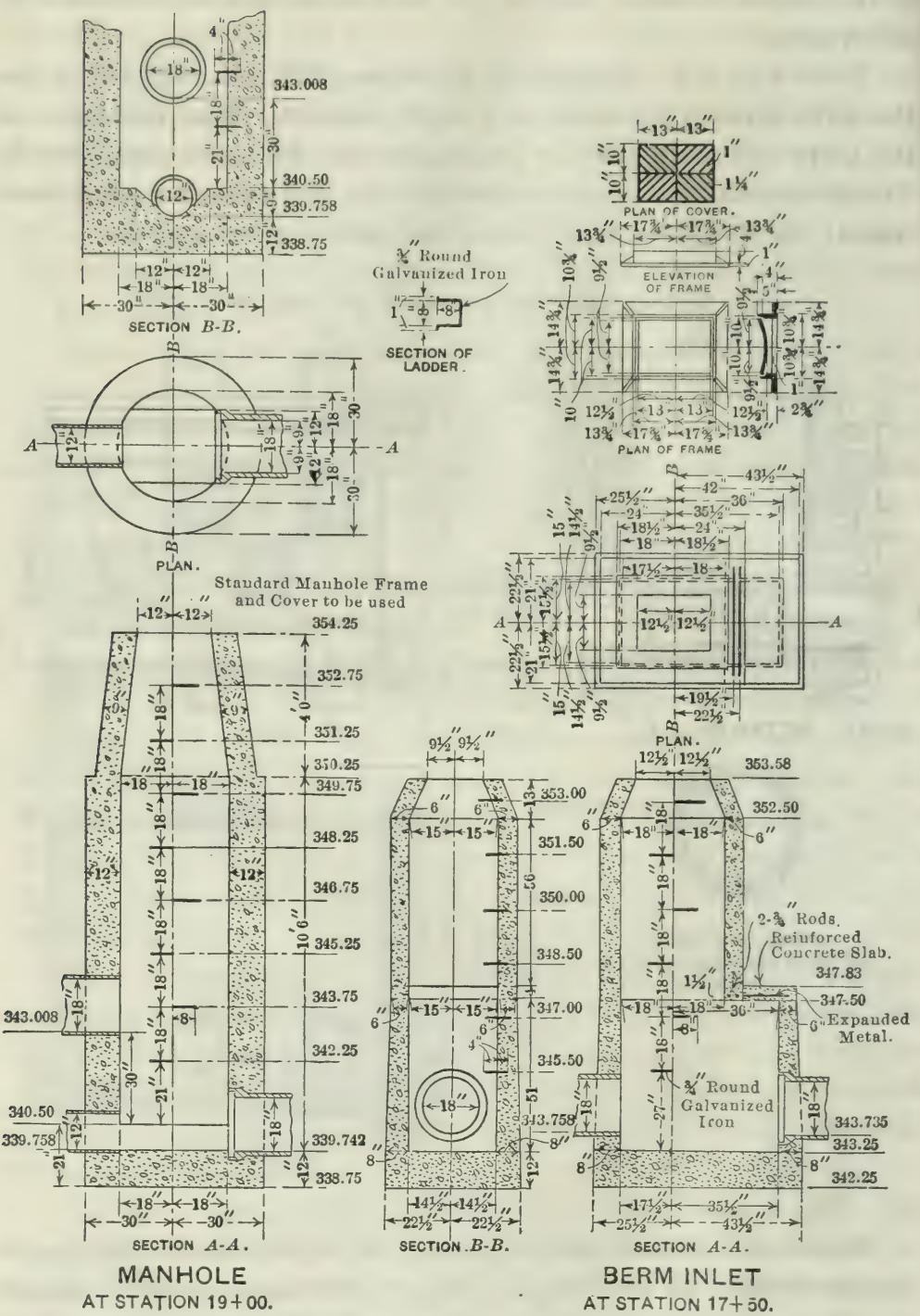

SECTION B-B.
PLAN.
Standard Manhole Frame and Cover to be used
SECTION A-A.
MANHOLE
AT STATION 19+00.
¾" Round Galvanized Iron
SECTION OF LADDER.
PLAN OF COVER.
ELEVATION OF FRAME
PLAN OF FRAME
PLAN.
SECTION B-B.
2-⅞" Rods, Reinforced Concrete Slab.
6" Expanded Metal.
¾" Round Galvanized Iron
SECTION A-A.
BERM INLET
AT STATION 17+50.

FIG. 11.

FIG. 12.—RIP-RAP ON DAM. RESERVOIR NEARLY FULL.

FIG. 13.—LOOKING SOUTHWARD ACROSS BASIN. RESERVOIR FULL.

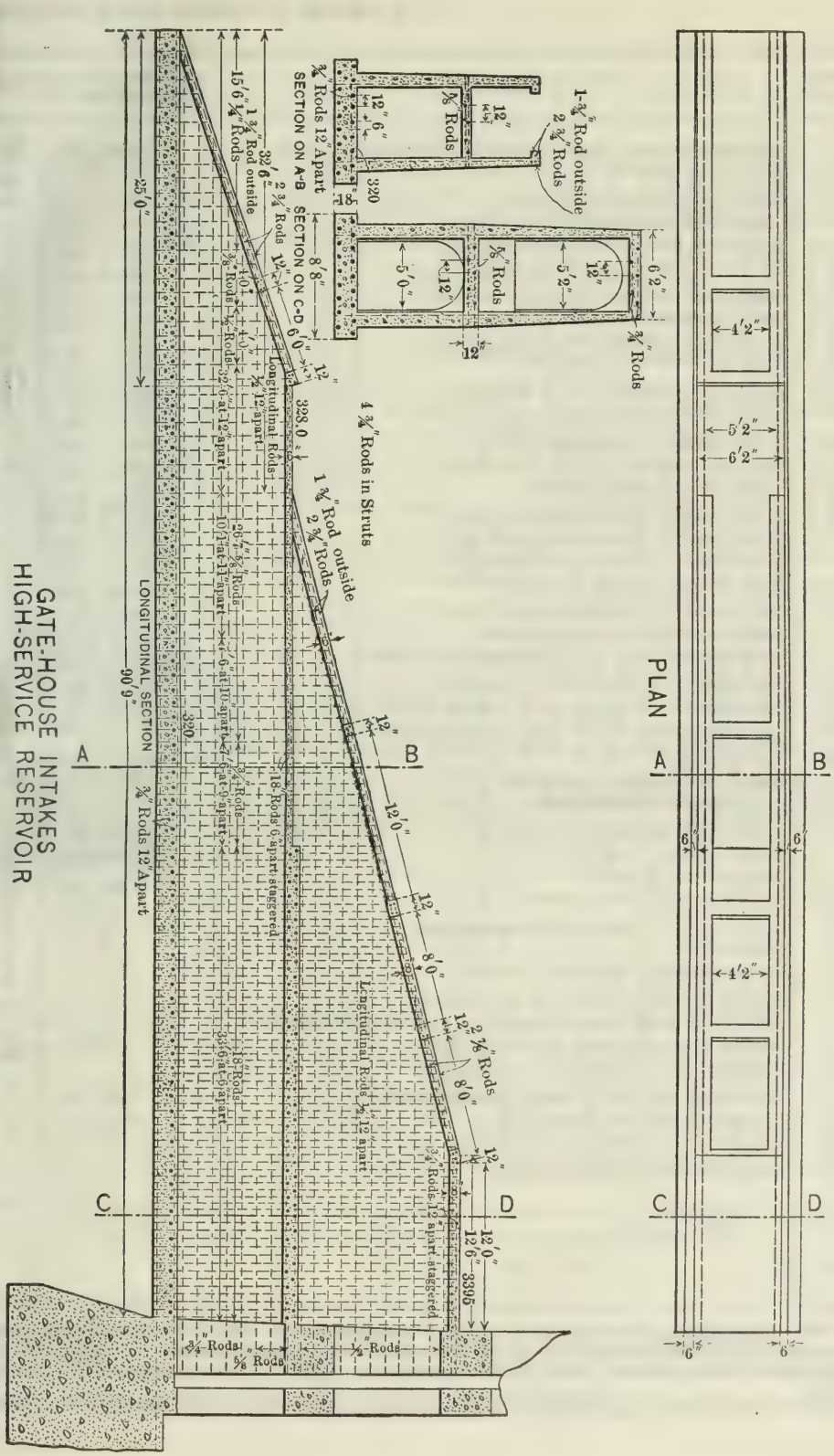

GATE-HOUSE INTAKES
HIGH-SERVICE RESERVOIR
LONGITUDINAL SECTION
PLAN
SECTION ON A-B
SECTION ON C-D
A
B
C
D
90'9"
25'0"
328.0
320
18
12"
12'0"
8'0"
6'0"
5'0"
6'2"
4'2"
5'2"
6"
1 3/4" Rod outside
2 3/4" Rods
3/4" Rods 12" Apart
4 3/4" Rods in Struts
Longitudinal Rods
1/2" Rods
3/4" Rods
5/8" Rods
2 7/8" Rods
18 Rods 6" apart staggered
3/4" Rods 12" apart staggered

Fig. 14.

TABLE 1.—Cost of Lake Ash-

Item No.	Items.	Total quantities.	Rate.
1	Stripping beneath dam, including boulders	13 500 cu. yd.	$0.38
2	Removing top soil	40 000 " "	0.35
3	Removing clay	80 000 " "	0.30
4	Earth excavation from basin	273 000 " "	0.20
5	Loose rock excavation from basin	125 000 " "	0.39
6	Solid rock excavation from basin	169 000 " "	0.82
7	Earth excavation, shallow trench	5 360 " "	0.40
8	Loose rock excavation, shallow trench	1 210 " "	0.80
9	Solid rock excavation, shallow trench	410 " "	2.00
10	Earth excavation, deep trench	14 000 " "	0.50
11	Loose rock excavation, deep trench	6 150 " "	1.10
12	Solid rock excavation, deep trench	260 " "	3.00
13	Back-fill	20 090 " "	0.18
14	Rolled or puddled clay in trench	5 000 " "	0.35
15	Rolled or puddled clay in cut-off wall	3 720 " "	0.35
16	Rolled or puddled clay in dam	35 000 " "	0.35
17	Rolled selected material in dam	152 000 " "	0.20
18	Random rock and earth in dam	95 000 " "	0.15
19	Top soil hauled and spread	30 000 " "	0.30
20	Top soil rolled and seeded	75 000 sq. yd.	0.03
21	Slope paving, Elevations 340 to 353	15 700 " "	1.60
22	Slope paving, Elevations 330 to 340	3 670 " "	1.80
23	Slope paving, Elevations 320 to 330	2 000 " "	2.00
24	Slope paving, below Elevation 320	2 000 " "	2.10
25	Plain concrete in cut-off wall	560 cu. yd.	8.00
26	Reinforced concrete in cut-off wall	2 060 " "	10.00
27	Plain concrete in bonding wall	210 " "	8.00
28	Tongued and grooved sheeting left in trench	100 000 ft. b. m.	50.00
29	Plain sheeting left in trench	80 000 " " "	40.00
30	Furnishing and laying 8-in. vitrified pipe	500 lin. ft.	0.40
31	Furnishing and laying 12-in. vitrified pipe	500 " "	0.60
32	Furnishing and laying 18-in. vitrified pipe	1 475 " "	1.00
33	Furnishing and laying 12-in. vitrified tile, including stone or gravel	1 490 " "	0.60
34	Storm-water inlets, complete	8	55.00
35	Gate-house, substructure	.....	.....
36	Gate-vault	.....	.....
37	Reinforced concrete conduit, 48-in	1 010 lin. ft.	7.00
38	Cement grout, 1 : 2	500 bags.	0.75
39	Neat cement grout	100 "	0.65
	Rock lining in dam	cu. yd.	0.15
	Scraper work)	" "	0.35
	Loose rock } East Drive excavation	" "	0.50
	Solid rock)	" "	1.00
	Extra yardage outside $2\frac{1}{2}$: 1 slope	" "	0.20
	Excess yardage, inlets	" "	8.00
	Excess material moved on 11th Street	.....	.....
	Allowed in west puddle trench to give total excavation	.....	.....
	Excess concrete, gate-house intakes	cu. yd.	6.50
	Excess reinforced concrete, intakes	" "	26.00
	Allowance on Slingluff Avenue fill	.....	.....
	Removing top soil on Park Board's property	cu. yd.	0.35
	Allowance on Park Board fill	.....	.....
	Fill, Beech Avenue and 7th Street	cu. yd.	0.40
	Dressing, not in original contract	.....	.....
	Miscellaneous force accounts	.....	.....
	Totals	.....	.....
	City credits	.....	.....

* The increased cost of the deep trench was due to the change in plan of the cut-off

† A saving of $10 000 was made at the dam due to the change in plan. The item $47 289.41 These prices were reduced until $10 000 were saved. The actual saving on the dam, inmates

§ This item is made up of three force accounts: Extra work on Liberty Road, $482.90;

BURTON HIGH-SERVICE RESERVOIR.

Estimated cost.	Total quantities.	Actual cost.	Increased cost.	Decreased cost.	Force accounts.	Total actual cost.
$5 130.00	13 978	$5 311.64	$181.64	.....	.....	$5 311.64
14 000.00	32 886	11 510.10	.....	$2 489.90	.....	11 510.10
24 000.00	63 846	19 153.80	.....	4 846.20	.....	19 153.80
54 600.00	161 307	32 261.40	.....	22 338.60		
48 750.00	98 752	38 513.28	.....	10 236.72	$4 878.63§	340 020.49
138 580.00	322 399	264 367.18	125 787.18	.....		
2 144.00	6 314	2 525.60	361.60	.....	.....	2 525.60
968.00	1 778	1 422.40	454.40	.....	.....	1 422.40
820.00	1 097	2 194.00	1 374.00	.....	.....	2 194.00
7 000.00	7 415	3 707.50	.....	3 292.50	.....	3 707.50
6 765.00	8 078	8 885.80	2 120.70	.....	.....	8 885.80
780.00	9 418	28 254.00	27 474.00*	.....	.....	28 254.00
3 616.20	2 725	490.50	.....	5 125.70	.....	490.50
1 750.00	3 910	1 368.50	.....	381.50	.....	1 368.50
1 302 00	25 032	8 761.20	7 459.70	.....	1 295.74	10 056.94
12 250.00	37 784	13 224.40	974.40	.....		
30 400.00	199 226	21 845.20	.....	8 554.80	1 649.26	47 289.41†
14 250.00	137 137	20 570.55	6 320.55	.....		
9 000.00	20 805	6 241.50	.....	2 758.50	.....	6 241.50
2 250.00	72 603	2 178.09	.....	71.91	.....	2 178.09
25 120.00	.....	.....	.....	16 295.73	8 824.27	8 824.27
6 606.00	.....	.....	.....	.....	.....	.....
4 000.00	.....	.....	.....	.....	.....	.....
4 200.00	.....	.....	.....	.....	.....	.....
4 480.00	60.81	486.48	.....	3 993.52	.....	486.48
20 000.00	.....	.....	.....	20 600.00	.....	.....
1 680.00	.....	.....	.....	1 680.00	.....	.....
5 000.00	.....	.....	.....	5 000.00	.....	.....
3 200.00	384 052	15 362.08	12 162.08	.....	.....	15 362.08
200.00	450	180.00	.....	20.00	.....	180.00
300.00	546.04	327.62	27.62	.....	.....	327.62
1 475.00	1 398.44	1 398.44	.....	76.56	.....	1 398.44
894.00	1 384	830.40	.....	63.60	.....	830.40
440.00‡	.....	540.00	100.00	.....	.....	540.00
28 600.00	.....	29 130.95	530.95	.....	1 306.72	30 437.67
2 150.00	.....	2 150.00	.....	.....	582.87	2 732.87
7 070.00	1 010	7 070.00	.....	.....	337.64	7 407.64
375.00	.....	.....	.....	375.00	.....	.....
65.00	.....	.....	.....	65.00	.....	.....
.....	36 448	5 467.20	5 467.20	.....	.....	5 467.20
.....	1 145	400.75	400.75	.....	.....	400.75
.....	50	25.00	25.00	.....	.....	25.00
.....	175	175.00	175.00	.....	.....	175.00
.....	1 200	240.00	240.00	.....	.....	240.00
.....	4.4	35.20	35.20	.....	.....	35.20
.....	.....	100.00	100.00	.....	.....	100.00
.....	.....	198.90	.....	.....	.....	198.90
.....	30.2	196.30	196.30	.....	.....	196.30
.....	47.7	1 240.20	1 240.20	.....	.....	1 240.20
.....	.....	1 200.34	1 200.34	.....	.....	1 200.34
.....	4 988	1 745.80	.....	.....	.....	1 745.80
.....	.....	2 500.00	2 500.00	.....	.....	2 500.00
.....	2 119	847.60	.....	.....	.....	847.60
.....	.....	.....	.....	.....	1 286.81	1 286.81
.....	.....	.....	.....	.....	3 734.77	3 734.77
$494 810.20	.....	.....	.....	.....	$23 896.71	$518 531.61
.....	.....	.....	.....	.....	.....	217.69
				Net total....	.....	$518 313.92

wall trench, and to the fact that more rock was found than was estimated.
should be $57 289.41 at the original prices for depositing the different materials in the dam.
cluding the force accounts and the rock lining, was $4 143.39 less than the original esti-
‡ A manhole was built which was not in the original contract.
sewer extension, $2 531.03; and drainage, springs, etc., $1 864.70.

Under normal conditions, the reservoir will act as an equalizer between the pumps at the Mt. Royal pumping station and the main on North Avenue, which supplies a large territory. Any surplus pumped, greater than the consumption, passes through the check-valve to the basin at the west end of the reservoir, the flap-valves in the side chambers preventing any discharge into the reservoir through these chambers and the intakes. If the pumpage is less than the consumption, or if the pumps are shut down, water is drawn through the side chambers, the check-valve preventing its return by way of the force-main. By this arrangement, circulation is obtained in the reservoir, as all water enters at the west end through the force-main and is drawn out through the intakes at the east end.

The excavated portion of the basin was hand-paved between Elevations 340 and 353 with rock from the shovels. On the dam, after the rock lining was dressed, rip-rap was laid as an additional protection to the slope.

In December, 1910, chloride of lime having been used freely, about 5 ft. of water were pumped into the basin and drawn off. On December 16th pumping recommenced, and was continued until January 11th, 1911, when the water surface reached Elevation 350.

As far as it is possible to determine, no water has passed through the dam, the insignificant quantity of seepage observed to date seeming to come entirely through the natural ground, and, with the exception of the springs led from the dam foundations through blind drains, being considerably removed from the toe of the dam.

The cost of the work is given in Table 1.

The work was designed by Emory Sudler, Assoc. M. Am. Soc. C. E., Assistant to Alfred M. Quick, M. Am. Soc. C. E., Water Engineer, and was executed under his supervision.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1243

A WESTERN TYPE OF MOVABLE WEIR DAM.*

By W. C. Hammatt, M. Am. Soc. C. E.

With Discussion by Messrs. Thomas C. Atwood and W. C. Hammatt.

Much has appeared recently, in the technical press and the publications of this Society, with regard to dams, due partly to the present activity in irrigation matters, and partly to the interest aroused by the failure of the Austin Dam. This paper, describing a dam of a kind rather common in California, but little known elsewhere, is presented with the hope of promoting profitable discussion and perhaps suggestions as to the construction of a weir of equal cheapness without the disadvantages of the type described.

This is a movable-weir dam with horizontal flash-boards and fixed bents. Its purpose is to maintain a uniform, or approximately uniform, level of water in a canal, river, or spillway, and have the ability to allow the free, or approximately free, passage of water in time of flood. Consequently, the type is used as diverting dams in rivers, as controlling and regulating gates in canals and spillways, and often even as head-gates for canals and ditches. From the nature of their use, these structures are generally located on soft and permeable foundations. Until the last few years they have generally been of timber, but the recent tendency toward concrete construction has also been felt. Their construction has largely been a growth of local or individual custom and experience, rather than of engineering design, and in many cases this is painfully apparent.

* Presented at the meeting of September 18th, 1912.

The writer will take up first the various forms of superstructure and foundations of the timber weirs, giving each a form letter, and afterward discussing them in detail.

The form shown in Fig. 1 will be called "Type A." In this the flash-boards move in a vertical plane between posts with guide-strips, nailed or bolted on, to hold them in place. The flash-boards are removed with grappling hooks hanging from tackle on head-frames over the boards. This type is often used as a combination wagon bridge and weir, and existing wagon bridges are sometimes converted into weirs.

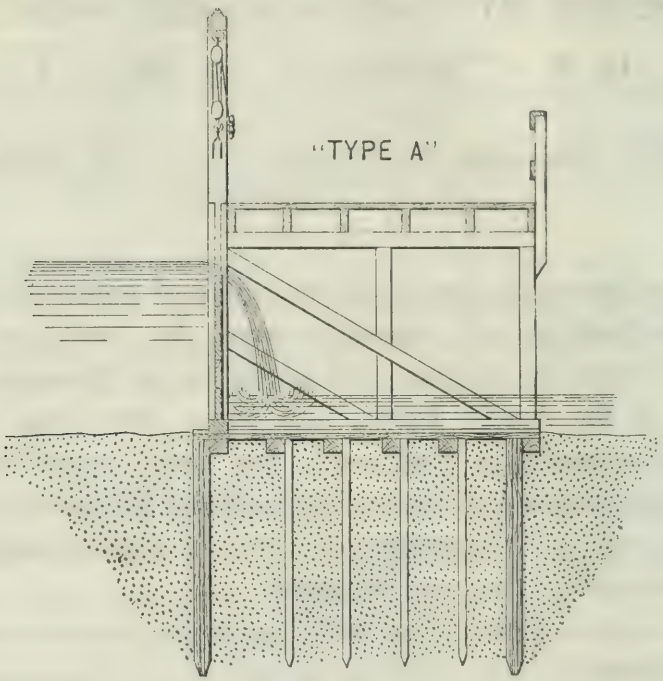


FIG. 1.

In "Type B," Fig. 2, the flash-boards are in a plane inclined down stream, moving in grooves formed by strips fastened to inclined timbers, or rafters. These boards are removed by men using pike-poles from the top of the weir, the incline enabling the operator to work a board upward until he can reach it without the aid of any elevating apparatus. This type, also, is sometimes used as a wagon bridge. Both types require an operating bridge above the high-water level for the storage of the flash-boards when not in use, and for a runway for operators while putting in and taking out the flash-boards.

Various methods are in vogue for anchoring these timber structures, and for shutting off the underflow. In some cases, they are placed on sills drift-bolted to standard piles, with one or more curtain-walls to cut off the underflow. In other cases, sheet-piling is used for both purposes, the sills being held in place by spiking sheet-piles to them.

The writer illustrates several systems for placing the cut-off walls which he has encountered. Figs. 1 and 2 show the floor system, consisting of mud-sills nailed to sheet-piling, with a continuous wall of equal depth above and below. This is sometimes varied by omitting the up-stream wall, sometimes by leaving gaps in the down-stream

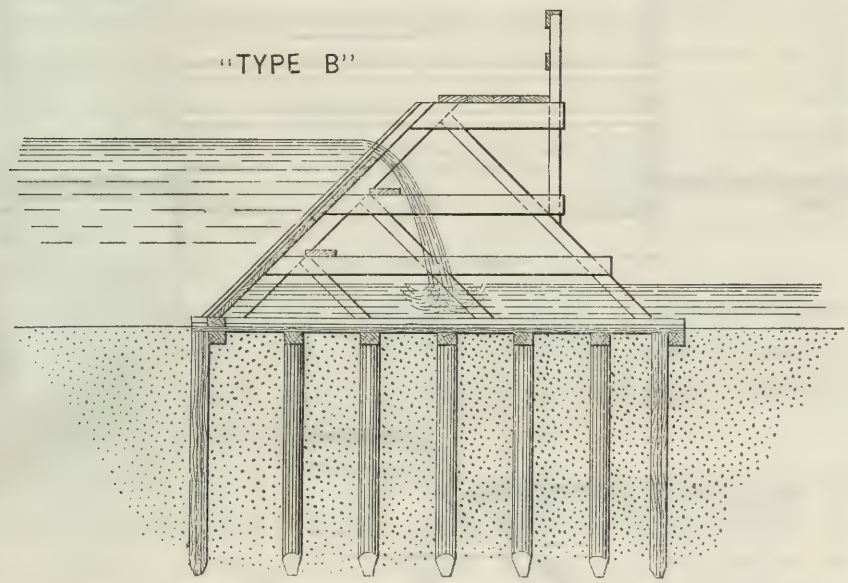


FIG. 2.

wall, and sometimes by boring holes in the floor to relieve the upward pressure. Occasionally, cross-walls are put in between the upper and lower cut-off walls at the bulkhead lines, and sometimes intermediate ones are put in. Fig. 3 shows a typical floor system on which the various cut-off walls are indicated.

Another device for overcoming the upward water pressure is shown in Fig. 4. It consists in laying a false floor from 4 to 6 ft. below the regular floor level, tying it to the regular floor by planking spiked to the sills, and filling the box thus formed with sand or gravel.

From an inspection of the forms illustrated, the following facts are apparent:

(1).—Since the form and material of the superstructure is such that its stability against translation and overturning depends on its attachment to its foundation, or anchorage, all forms fail if conditions are such that the foundation is intermittently wet and dry. Under this condition, the life of the timber is very short, and the fastenings give way as soon as rot starts, whereupon the structure inevitably

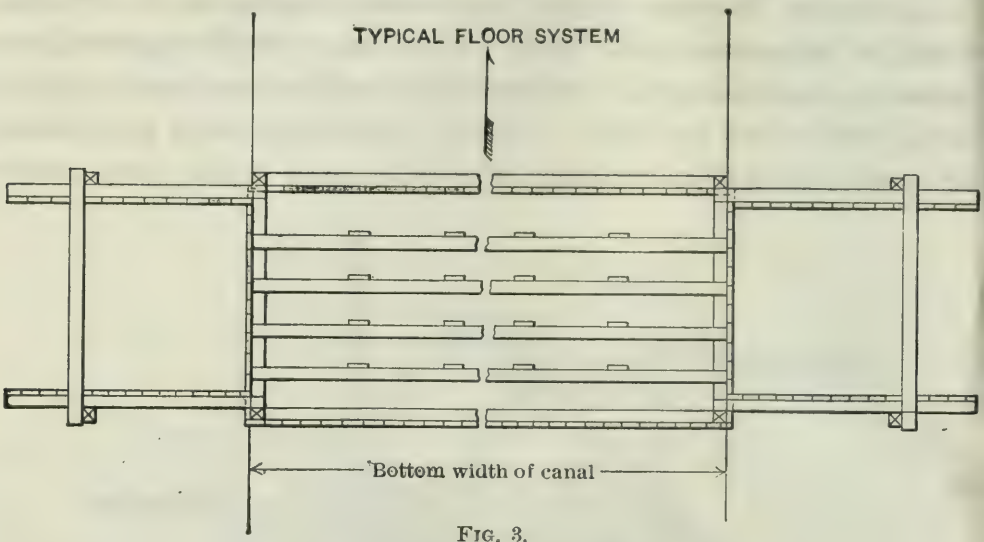


FIG. 3.

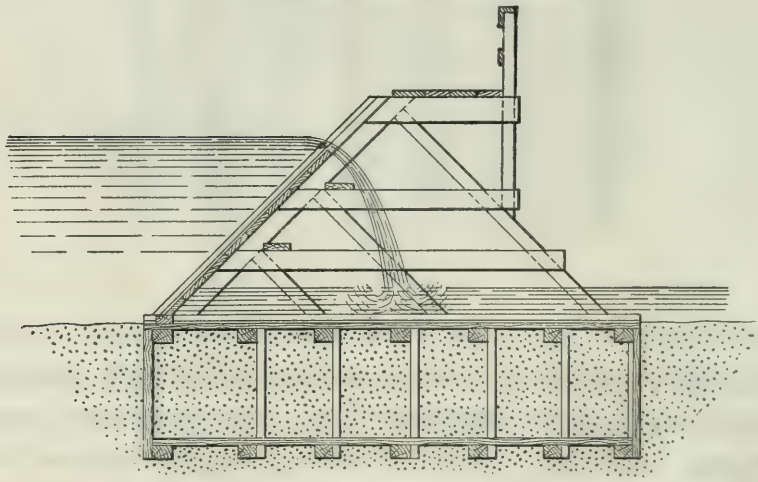


FIG. 4.

fails either by sliding or rising, according to whether or not the water has passed the upper cut-off wall.

(2).—If the foundation, or floor system, is always covered by water, the life of the timber is practically permanent, and the safety of the structure depends on the resistance of the fastenings to the deterioration of time. As these fastenings can be, and generally are, made

with a sufficient excess of material to withstand successfully the attacks of rust indefinitely, the matter of stability can be considered without regard to the life of the materials.

Let it be supposed that the soil in the bottom of the canal or river is fairly permeable, as is nearly always the case, and consider "Type A" of superstructure, with a water-tight floor and two water-tight cut-off walls of equal depth. According to the testimony of Mr. W. G. Bligh, as to experiments made in India, the loss of pressure due to a cut-off wall is equal to that of a base of double the length of the depth of the cut-off wall. Under this assumption, a calculation of the pressures acting both against the face of the weir and below the floor will show that there is an uplift on the structure which is greatest at the up-stream face and diminishes toward the down-stream end of the apron, which uplift is resisted only by the skin friction of the sheet-piling and the fastening of the superstructure thereto. This uplift is greatest in "Type A," and, in "Type B," diminishes with the angle of the weir face to the horizontal plane, because the pressure against the weir face approaches the vertical as the plane of the weir face approaches the horizontal. The uplift is greatest at the up-stream cut-off wall. The uplift at the down-stream edge of the apron is least when there is no cut-off wall at this point, and increases with the depth of the cut-off at that place.

This tendency to uplift is often resisted by extending the floor a little up stream from the face of the weir and weighting it with earth. This is generally sufficient to guard the up-stream end, so that the failure is nearly always by pulling the down-stream piles. If the bent does not extend the whole length of the floor, in a well-braced triangular form, the failure is occasionally by upheaval of the intermediate floor.

Destructive criticism is of little value unless accompanied by the remedy; therefore, although there is little to recommend the timber structures but their cheapness, the writer will suggest a method of construction which will make them less objectionable. Fig. 5 shows a general plan which he proposes as a substitute for the unstable structures just described. This combines a simplicity and cheapness nearly equal to those obtained in them, with conditions of greater stability. The bents, *B, C, D,* should be founded on standard piles, driven to sufficient depth to give the necessary bearing value as well

as safety against undermining. *J F* is the main cut-off wall, which should be of sheet-piling driven to sufficient depth and with sufficiently tight joints to reduce the velocity of the seepage flow to less than that which would carry with it the foundation material. Triple-lap, tongued and grooved, or some such piling should be used, and great care should be taken not to split it in driving and thus destroy its imperviousness. Another row of sheet-piling, *A E*, not necessarily as deep, may be driven up stream from the first, and a battened floor, *A J*, may be laid between the two rows. This floor need not rest on piling, but may be laid on mud-sills, if the bottom soil is thoroughly compact. Between the bents, there should be an apron, *J K*, of open

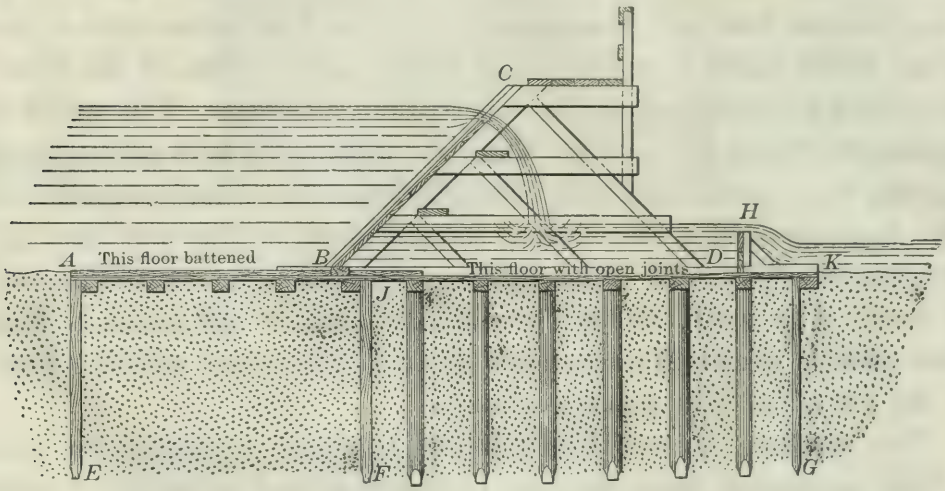


FIG. 5.

design, in order to avoid under pressure, and extending down stream a sufficient distance to protect the bottom against erosive action. At or near the end of the apron, another line of sheet-piling, *K G*, may be driven, not necessarily of great length or tightness, but merely as a precaution against undermining. If the fall is greater than 4 ft., a small bulkhead, *H*, should be built across the floor a few feet back from its lower edge, in order to form a pool to act as a water cushion. Instead of this, the floor may be built about 2 ft. lower than the bed of the stream or canal, but the former is the simpler construction, especially where the lower floor level might cause great additional inconvenience and expense of construction.

If the foregoing design is used, and the construction is carefully superintended, special care being taken to keep *A E*, *A J*, and *J F*

water-tight, and J K with free passage for the seepage flow, there will always be a pressure on the piles instead of a pull, and consequently less likelihood of failure.

The cost of these structures is comparatively small. The diverting dam of the San Joaquin and Kings River Canal and Irrigation Company, across the San Joaquin River, near Mendota, Cal., 16 ft. in height and 380 ft. in clear width, passing 40 sec-ft. of water per ft.

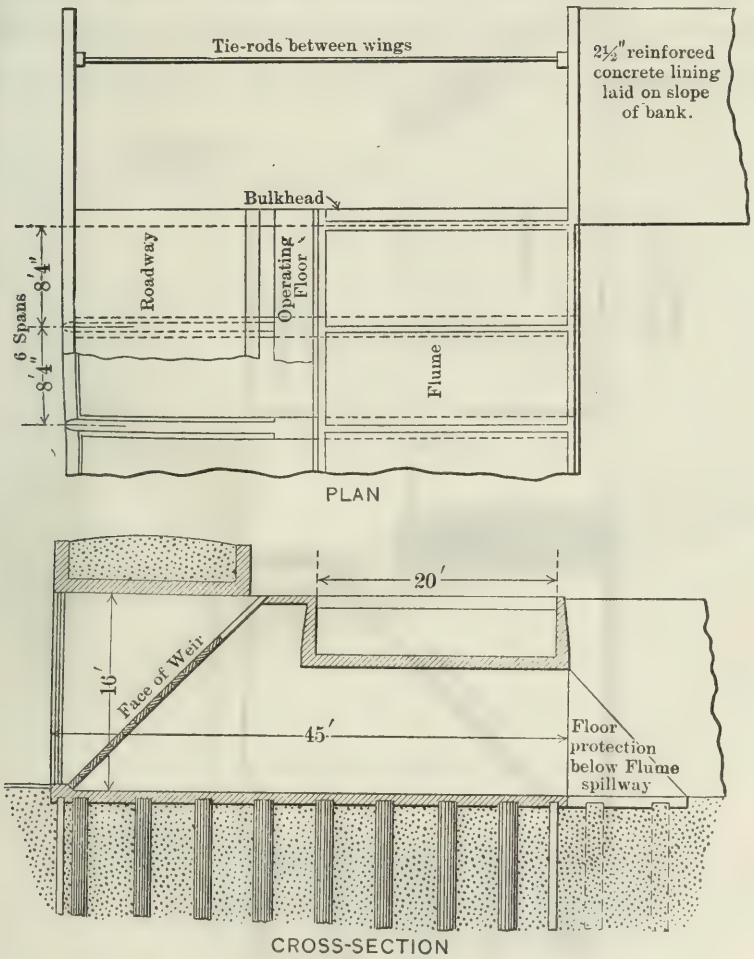


FIG. 6.

of width when open, cost about $140 per ft. of width. Records of cost of smaller structures in canals and sloughs, passing from 6 to 50 sec-ft. of water per ft. of width, show a total cost of from $25 to $100 per ft. of width, according to size and difficulty of construction.

Recently, there has been a tendency toward concrete construction. With the going out of use of timber for irrigation structures, the type

itself is disappearing, being replaced in general by regulators with submerged openings. There have been a few cases, however, where the material of construction has been changed without essentially changing the design. Among the latter are two reinforced concrete weirs, one at the head of Temple Slough, Merced County, California, and

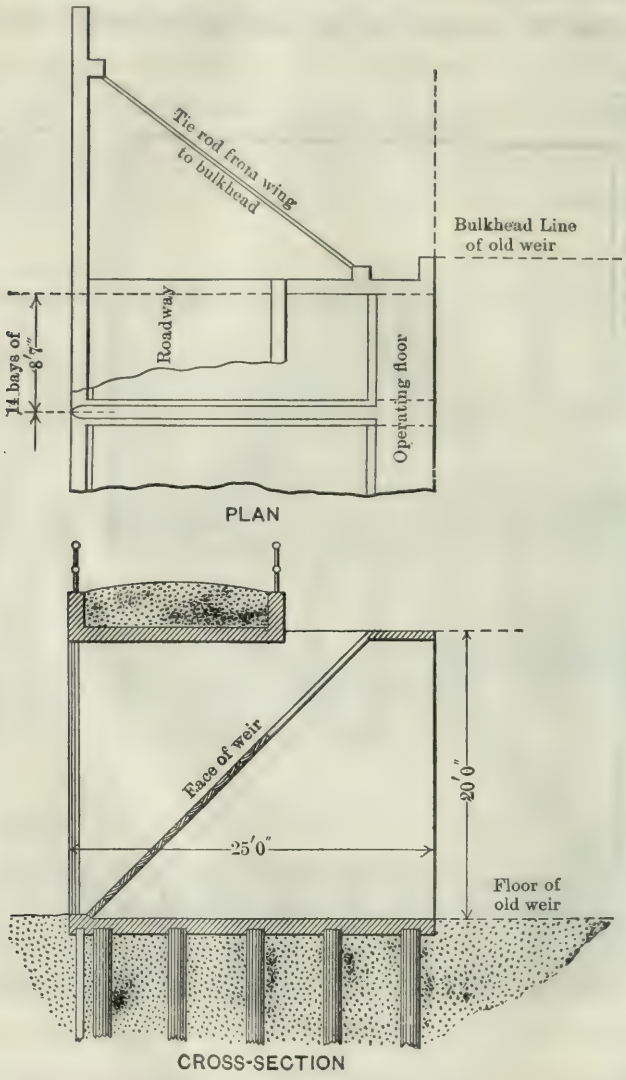


FIG. 7.

the other in the Kern Valley Water Company's canal, in Kern County, California, both designed by John B. Leonard, M. Am. Soc. C. E., for Miller and Lux, Incorporated. The former carries the Poso Canal across in a flume, and is of the dimensions and form approximately given in Fig. 6. The one in Kern County is shown in

ig. 7, and relies on the bulkheads and floor of the old timber struc-
ıre below for bank and bottom protection, which accounts for the
ıort length of the apron. It will be noted that the uplift in the up-
ream toe, in both cases, is counterbalanced by a reinforced concrete
ridge carrying a roadway.

The writer sees nothing to criticize in these designs, for they are
s strong and stable structures as could be built of this type. Some
xpense might be saved, however, by putting weep holes in the apron,
o relieve the pressure, and by extending this apron up stream from
he up-stream weir face to reduce the gradient of seepage flow.

DISCUSSION

Mr. Atwood.

THOMAS C. ATWOOD, M. AM. SOC. C. E.—The author has performed a distinct service by bringing before the Society the subject of inexpensive movable dams for use in irrigation canals, small streams, etc., and in pointing out correct principles of design.

As he well states, "Their construction has largely been a growth of local or individual custom and experience, rather than of engineering design, and in many cases this is painfully apparent." It is to be hoped that the recent agitation for a stricter supervision of dams, following the failure of that at Austin, Pa., may extend even to such small structures as these, and that all of them may be placed in the hands of competent engineers.

This type of dam seems to be better adapted to irrigation canals than to streams, due to the likelihood of driftwood in the latter lodging against the bents in time of flood and holding back the water, in spite of the removal of the flash-boards, and, perhaps, causing the loss of the structure. For such locations, especially where the dam is a high one, as the diverting dam of the San Joaquin and Kings River Canal and Irrigation Company across the San Joaquin River, where the height is given as 16 ft., a type of movable dam which gives a clear waterway when lowered is usually to be preferred. Many such dams have been built by the United States Army Engineers, and have proved very successful. Although permanent in character, the cost of the waterway is not excessive, the principal expense being in the provision for navigation. The cost of the navigable pass of Dam No. 26, on the Ohio River, is given by the U. S. Engineers' office as about $175 per ft., the dam being built on a rock foundation, and for a maximum head of 14.2 ft. Mr. J. W. Arras, U. S. Assistant Engineer, gives the probable cost of a Chanoine dam on piles in a gravel foundation as from $250 to $300 per ft. for a structure about 16 ft. in height, according to local conditions.

The failure of Dam No. 26 was due apparently to the failure of the rock foundation, although the unwatering of the site may reveal some other cause. This is worthy of note, in view of the almost uniform success of these dams when founded on the river gravel.

The author does well to lay stress on the importance of the foundation, and his suggested design, as shown in Fig. 5, is excellent in this respect on most points.

The subject of scour does not seem to be adequately treated, however, although Mr. Hammatt called attention to this point in his discussion* on the Yuma River Débris Barrier, where he suggested an unattached apron of large concrete blocks which would follow the bottom down, as scour takes place below the dam, and prevent the

* *Transactions*, Am. Soc. C. E., Vol. LXXI, p. 229.

gravel under the dam itself from being washed away. This method is excellent, and has been used successfully, as have a number of others, as described by the writer in discussing the paper on the Yuma River Débris Barrier mentioned above. Mr. Atwood.

The suggested design, as given in Fig. 5, is weak in this respect, as scour may reasonably be expected when the flash-boards are raised and the direct current strikes the small bulkhead, *H*, and jumps over it. This can be helped by moving this bulkhead nearer to the main dam, extending the plank apron farther down stream, or placing rip-rap below the apron to prevent scour and follow the bottom down if any occurs.

The best method, however, and probably the cheapest, is to slope the apron upward in the direction of the flow, with perhaps 2 ft. rise in 20 ft., giving the same depth of water just below the dam, but doing away with the bulkhead, *H*. There will still be scour, but it will be transferred to such a distance below the dam that the foundation of the structure will not be endangered, the tendency of the eddy being to bring gravel back toward the apron, rather than to scour that immediately adjacent.

W. C. Hammatt, M. Am. Soc. C. E. (by letter).—The writer is glad if his paper has added anything to the knowledge of small inexpensive structures for canals, etc., but does not wish to be understood as recommending the use of timber as a construction material, except in the extremely rare occasions of economic utility. Under ordinary conditions, the cost of reinforced concrete is only 50% more than the equivalent timber construction, and this should generally make its use preferable. Mr. Hammatt.

Mr. Atwood states that this type of dam is more adapted to use in irrigation canals than in streams, due to the likelihood of driftwood in the latter lodging against the bents in time of flood. As a matter of fact, dams of this type are used extensively in rivers, the drift being taken care of by floating log booms arranged so as to conduct it into still water. On the other hand, structures of any type having removable parts which are seated in grooves or sockets in the floor are found to be impracticable in Western rivers because of the deposition of sand on the floor on the recession of the flood. In the case of the flash-board weir, this sand is scoured by the increased velocity on the approach of the board as it is dropped in the grooves, and the board seats itself in a few minutes, regardless of the quantity of sand which has been deposited.

In the rivers of the West the flow conditions differ from those of the East. For example, the flow in the San Joaquin River ranges from about 200 sec-ft. minimum to more than 50 000 sec-ft. maximum. The sudden termination of one of these floods will cause the deposit of sometimes as much as 3 ft. of sand on the floor of the weir.

Mr. Hammatt.

Regarding the question of scour: In general, the weir floor is made long enough so that there is not much tendency in that direction. If the length of the floor is such that a line from the top of the weir, making an angle of 45° with the horizontal, falls within the limits of the floor, there is little tendency to scour. Generally, there is no occasion to put in the small auxiliary bulkhead shown in the writer's plan, as a sand-bar almost always forms a short distance below the weir, which has the same effect, as it backs up the water and forms a water-cushion to break the impact of the overpour. The same effect is obtained by placing the elevation of the weir floor 1 or 2 ft. below the grade of the canal or river bottom.

The expedient suggested by Mr. Atwood, namely, that of sloping the floor upward, would have an effect the opposite of that desired. The velocity of a certain volume of water falling over a vertical weir is less than that of the same volume falling over a weir inclined down stream. Consequently, the scour would be less in the former case.

In conclusion, it may be stated that this type of structure, built of the material which will give the maximum service, and with the substructure best adapted to the foundation on which the weir is to stand, is probably the least expensive of any of the movable types for waterways where the floods are flashy and a large, and at the same time accurate, regulation is desired.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1244

A SHORTENED METHOD IN ARCH COMPUTATION.

By H. A. Sewell, Esq.

With Discussion by William Cain, M. Am. Soc. C. E.

In the design of elastic arches, as given by William Cain, M. Am. Soc. C. E., the method of single loads is more accurate than that of resultant moment polygon, because it computes the moment and thrust for each load separately, while the latter computes these quantities for all the loads together. Thus, in computing

$$\Sigma\,(mb'\,y) = \frac{\Sigma\,(bh)\;\Sigma\,(y)}{N} - \Sigma\,(bh'\,y),$$

the two quantities in the right member of the equality are so nearly equal in the latter method that they must be multiplied out by long hand, hence multiplying errors; while in the former the quantities dealt with are so much smaller that an ordinary slide-rule will usually suffice, thus eliminating false accuracy.

On the other hand, the polygon method requires the computation of, at most, only six polygons, corresponding to different positions of the live load; while the method of single loads requires as many polygons as there are loads to the left of the crown, although these latter are somewhat easier to compute.

Because of its greater accuracy, and because it determines the exact position of the live load for maximum moment at any given section, rather than assumes its arbitrary position for maximum moments, the single-load method, doubtless, would be much more widely used,

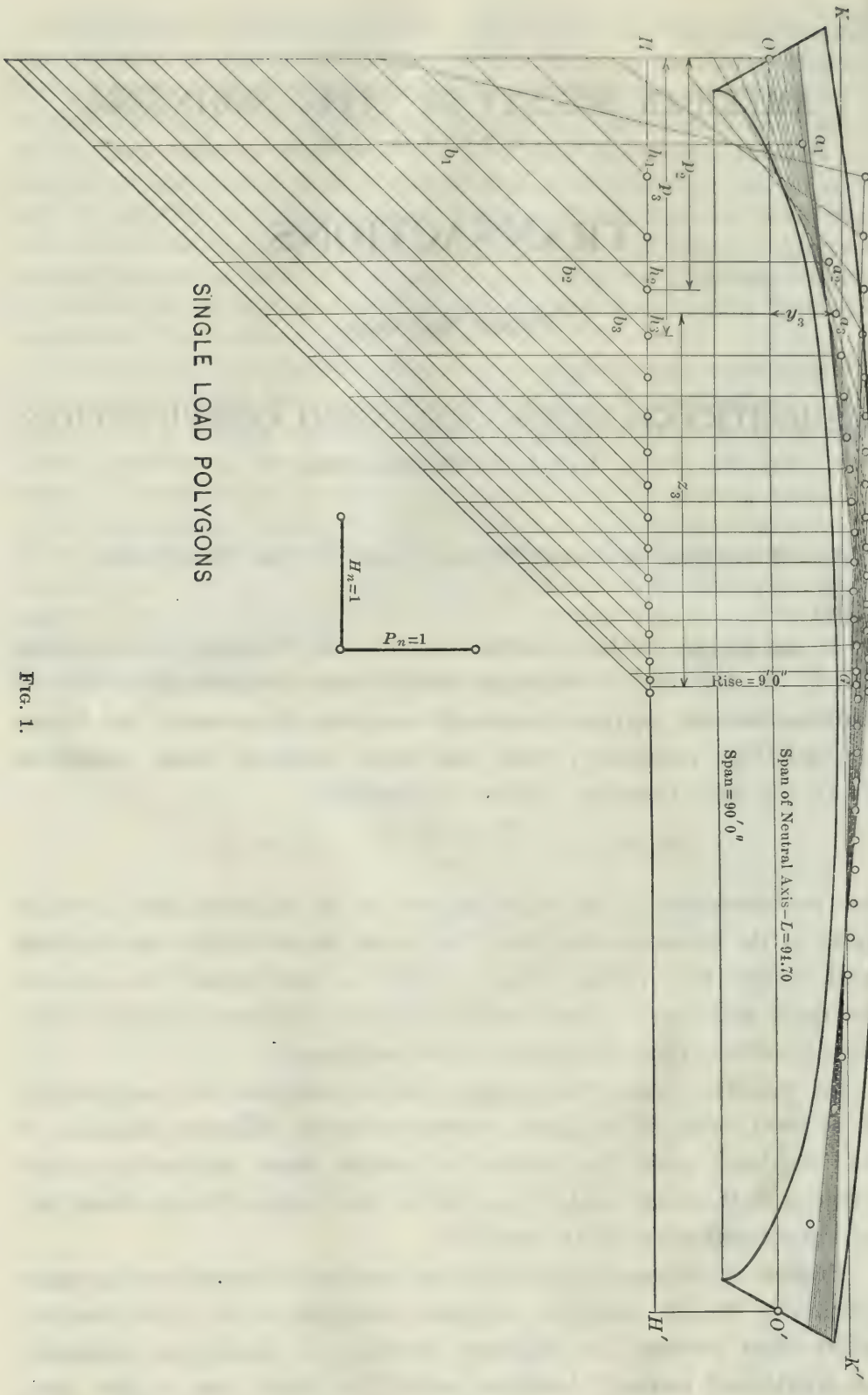

SINGLE LOAD POLYGONS
$H_n=1$
$P_n=1$
Span of Neutral Axis—$L=94.70$
Span = 90′ 0″
Rise = 9′ 0″

FIG. 1.

especially for very flat or very high arches, were it not for the much greater labor involved. However, in the following method, the computations are greatly reduced, while, at the same time, a check is introduced, which cuts the labor of computing moments again by making the computations self-checking.

The summations, $\Sigma\ (y)$, $\Sigma\ (y^2)$, $\Sigma\ (z)$, and $\Sigma\ (z^2)$, having been computed for the arch ring by the usual method, the quantities, $\Sigma\ (bh)$, $\Sigma\ (bh'\ z)$, and $\Sigma\ (bh'\ y)$, should next be obtained for each of the trial polygons corresponding to loads unity and horizontal thrusts unity at each of the load points, P_n.

The force polygon for load unity and horizontal thrust unity is taken so that the pole point is one unit horizontally to the left of the lower extremity of the load vector unity, thus making the right component of each trial-moment polygon horizontal and the left component inclined downward at an angle of 45 degrees. By reference to Fig. 1, the following relations are discovered:

$$\sum_o^n (bh) = \sum_o^{n-1} (bh) + (n-1)\,(p_n - p_{(n-1)}) + \left(p_n - \frac{L}{2} + z_n\right)$$

$$\sum_o^n (bh'\ z) = \sum_o^{n-1} (bh'\ z) + (p_n - p_{(n-1)}) \sum_o^{n-1} (z) + \left(p_n - \frac{L}{2} + z_n\right) z_n$$

$$\sum_o^n (bh'\ y) = \sum_o^{n-1} (bh'\ y) + (p_n - p_{(n-1)}) \sum_o^{n-1} (y) + \left(p_n - \frac{L}{2} + z_n\right) y_n$$

in which:

L = span of the neutral axis;
y = ordinate at a of arch from horizontal through spring, OO';
z = distance of a to left of crown, G;
p = distance of load considered from left spring, O;
n = number of load considered from left;
bh = ordinate of polygon from horizontal, HH'.

Thus, each summation is made to depend on the one preceding it and the quantities $(p_n - p_{(n-1)})$ and $\left(p_n - \frac{L}{2} + z_n\right)$. The work of these computations on a hypothetical flat arch is shown in Table 1.

TABLE I.

Point	(z)	(y)	(z^2)	(y^2)	Load	p	$\frac{L}{2}-p$	$p_n-p_{(n-1)}$	$\Sigma_o^{n-1}(z)$	$\Sigma_o^{n-1}(y)$	$\Sigma\,(bh)$	$\Sigma(bh'z)$	$\Sigma(bh'y)$
a_1	40.80	2.50	1664.0	6.25	P_1	8.90	38.45	8.90			2.35	96.0	5.88
					P_1'	13.42	33.93	4.52			6.87	280.5	17.18
a_2	31.97	4.43	1022.0	19.62	P_2	17.35	30.00	3.93	40.80	2.50	3.93	160.5	9.83
									31.97	4.43	1.97	63.0	8.73
											12.77	504.0	35.74
a_3	28.13	4.95	791.5	24.50	P_3	20.88	26.47	3.53	72.77	6.93	7.06	257.0	24.46
									28.13	4.95	1.66	46.7	8.22
											21.49	807.7	68.42
a_4	24.84	5.25	617.0	27.55	P_4	24.04	23.31	3.16	100.90	11.88	9.48	318.6	37.52
									24.84	5.25	1.53	38.0	8.03
											32.50	1164.3	113.97
a_5	21.82	5.52	496.0	30.50	P_5	26.96	20.39	2.92	125.74	17.13	11.68	367.0	50.05
									21.82	5.52	1.43	31.2	7.90
											45.61	1562.5	171.92
a_6	19.02	5.75	361.8	33.05	P_6	29.64	17.71	2.68	147.56	22.65	13.40	395.3	60.70
									19.02	5.75	1.31	24.9	7.53
											60.32	1982.7	240.15
a_7	16.47	5.92	271.2	35.05	P_7	32.13	15.22	2.49	166.58	28.40	14.94	415.0	70.80
									16.47	5.92	1.25	20.6	7.41
											76.51	2418.3	318.36
a_8	14.00	6.05	196.0	36.60	P_8	34.54	12.81	2.41	183.05	34.32	16.87	441.5	82.78
									14.00	6.05	1.19	16.7	7.20
											94.57	2876.5	408.34
a_9	11.67	6.16	136.0	37.95	P_9	36.83	10.52	2.29	197.05	40.37	18.32	451.3	92.45
									11.67	6.16	1.15	13.3	7.09
											114.04	3341.1	507.88
a_{10}	9.42	6.25	88.7	39.10	P_{10}	39.04	8.31	2.21	208.72	46.53	19.89	461.5	102.95
									9.42	6.25	1.11	10.5	6.95
											135.04	3813.1	617.77
a_{11}	7.25	6.32	52.6	39.95	P_{11}	41.18	6.17	2.14	218.14	52.78	21.40	467.0	113.00
									7.25	6.32	1.08	7.8	6.83
											157.52	4287.9	737.60
a_{12}	5.14	6.37	26.4	40.60	P_{12}	43.27	4.08	2.09	225.39	59.10	22.99	471.0	123.50
									5.14	6.37	1.06	5.4	6.76
											181.57	4764.3	867.86
a_{13}	3.06	6.39	9.4	40.80	P_{13}	45.33	2.02	2.06	230.53	65.47	24.72	475.0	134.80
									3.06	6.39	1.04	3.2	6.65
											207.33	5242.5	1009.31
a_{14}	1.01	6.40	1.0	40.90	P_{14}	46.85	0.50	1.52	233.59	71.86	19.76	355.2	109.20
									1.01	6.40	0.51	0.5	3.26
											227.60	5598.2	1121.71
Σ_0^{14}	234.60	78.26	5713.6	452.42	P_{15}	47.85	−0.50	1.00	234.60	78.26	14.00	234.6	
											241.60	5832.8	

In order to make the use of the formulas clear, a case is taken, $P_n = P_3$, from Table 1. Then for $P_{(n-1)} = P_2$, $\Sigma\ (bh) = 12.77$, $\Sigma\ (bh'\ z) = 504.0$, and $\Sigma\ (bh'\ y) = 35.74$; and, by adding the values of $z_{(n-1)} = z_2$, and $y_{(n-1)} = y_2$ to the last totals, we obtain $\sum_{o}^{n-1} (z) = 72.77$ and $\sum_{o}^{n-1} (y) = 6.93$. Multiply these latter quantities by the value of $(p_n - p_{(n-1)}) = 3.53$ and place the products under the values of $\Sigma\ (bh'\ z)$ and $\Sigma\ (bh'\ y)$ given above; and multiply $(p_n - p_{(n-1)})$ by $(n - 1) = 2$, placing the result under $\Sigma\ (bh)$. Next obtain $\left(p_n - \frac{L}{2} + z_n\right) = z_n - \left(\frac{L}{2} - p_n\right) = 1.66$, and place it in the $\Sigma\ (bh)$ column; then multiply it by $z_n = 28.13$ and $y_n = 4.95$, placing the results in the $\Sigma\ (bh'\ z)$ and $\Sigma\ (bh'\ y)$ columns. Finally, add the quantities below the last addition, in order to obtain the totals, $\Sigma\ (bh) = 21.49$, $\Sigma\ (bh'\ z) = 807.7$, and $\Sigma\ (bh'\ y) = 68.42$.

The check on the totals is shown by $\Sigma\ (z)$ being equally distant from $\Sigma\ (bh)$ for the loads, P_{14} and P_{15}. Likewise, $\Sigma\ (z^2)$ should be midway between $\Sigma\ (bh'\ z)$ for the loads P_{14} and P_{15}. No check was discovered for $\Sigma\ (bh'\ y)$ except to compute $\Sigma\ (bh'\ y)$ for Point P_{14} independently by the usual method. $\Sigma\ (z)$ and $\Sigma\ (y)$ are checked by direct addition. Checking the totals for P_{14}, checks all the others, because they are all carried forward in making the totals. All the multiplications may be made with an ordinary slide-rule.

These summations being obtained, the work is carried forward in the usual manner, as outlined by Professor Cain.

DISCUSSION

Mr. Cain.

WILLIAM CAIN, M. AM. SOC. C. E. (by letter).—The author has made a successful attempt to shorten certain calculations pertaining to the theory of the arch without hinges, where single loads are considered. As the number of parts into which the arch ring is divided is increased, the burden of computation is very much increased, and any device for shortening the work will be appreciated by the computer.

As the subject is of such practical importance, the writer will give a method of computing the quantities in question by a brief and independent procedure, and will incidentally derive check formulas for the difference method proposed by the author.

In the diagram, Fig. 2, the horizontal distances of the loads, P_1, P_2, . . . , from the center of the span, A, will be denoted by d_1, d_2, ...

From any P, as P_2, are drawn the two sides of the trial equilibrium polygon pertaining to this load, the one to the right of the load, $P_2\ A$, being horizontal, the one to the left, $P_2\ B_2$, being inclined at an angle of 45° to the horizontal. It follows that $b_4\ h_4 = h_4\ P_4$, $b_4\ h_3 = h_3\ P_4$, etc.; whence, for the lines, $P_4\ B_4$, $P_3\ B_3$, we have,

$$\begin{aligned} b_4\ h_4 &= z_4 - d_4 \\ b_4\ h_3 &= z_3 - d_4 \\ b_4\ h_2 &= z_2 - d_4 \\ b_4\ h_1 &= z_1 - d_4 \end{aligned}$$

$$\sum_o^4 b_4\ h = \sum_o^4 (z) - 4\ d_4,$$

$$\begin{aligned} b_3\ h_3 &= z_3 - d_3 \\ b_3\ h_2 &= z_2 - d_3 \\ b_3\ h_1 &= z_1 - d_3 \end{aligned}$$

$$\sum_o^3 b_3\ h = \sum_o^3 (z) - 3\ d_3.$$

Or, more generally, dropping the b subscripts, the sums can be indicated thus:

$$\sum_o^n (bh) = \sum_o^n (z) - n\ d_n \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Thus, for $n = 4$, referring to the load, P_4, and the side of its equilibrium polygon, $P_4\ B_4$.

$$\sum_o^4 (bh) = b_4\ h_4 + b_4\ h_3 + b_4\ h_2 + b_4\ h_1; \qquad \sum_o^4 (z) = z_1 + z_2 + z_3 + z_4.$$

This formula enables us to compute directly $\sum_o^n (bh)$ corresponding to $P_n\ B_n$ for any P_n, or for $n = 1, 2, 3, \ldots$ in turn.

Now let,

$$\sum_o^n (bh.z) = bh_1.z_1 + bh_2.z_2 + bh_3.z_3 + \ldots + bh_n.z_n.$$

Mr. Cain.

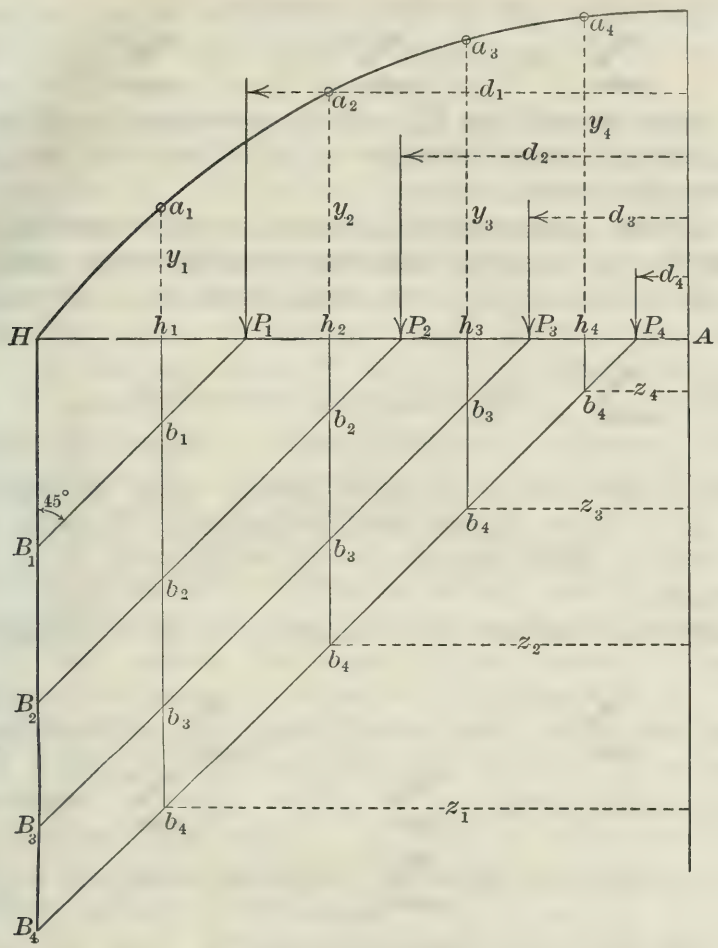


FIG. 2.

Thus, if $n = 4$, the bh's refer to the line, $P_4\ B_4$. It follows, from the relations given above, that,

$$\sum_o^n (bh.z) = (z_1 - d_n)\, z_1 + (z_2 - d_n)\, z_2 + \ldots + (z_n - d_n)\, z_n$$
$$= (z_1^2 + z_2^2 + \ldots + z_n^2) - d_n\,(z_1 + z_2 + \ldots + z_n)$$

or,

$$\sum_o^n (bh.z) = \sum_o^n (z^2) - d_n \sum_o^n (z) \quad \ldots\ldots\ldots (2)$$

in which the notation is sufficiently explained by the equivalent expansions above. Similarly, if,

$$\sum_o^n (bh.y) = bh_1.y_1 + bh_2.y_2 + \ldots + bh_n.y_n,$$

indicates the sum of quantities of the type $(bh.y)$ for any line $P_n\ B_n$ corresponding to a load, P_n, we have,

$$\sum_o^n (bh.y) = (z_1 - d_n)\, y_1 + (z_2 - d_n)\, y_2 + \ldots + (z_n - d_n)\, y_n$$
$$= (z_1\, y_1 + z_2\, y_2 + \ldots + z_n\, y_n) - d_n\,(y_1 + y_2 + \ldots + y_n)$$

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which can be indicated by the shorter notation,

$$\sum_{o}^{n} (bh.y) = \sum_{o}^{n} (z\,y) - d_n \sum_{o}^{n} (y) \ldots\ldots\ldots (3)$$

Thus the sums desired, for P_n and its corresponding $P_n\ B_n$, can be computed directly from Formulas (1), (2), and (3), on giving the proper numerical value to n.

Before giving a numerical application of the formulas, it may be well to derive the author's difference formulas from them.

Thus, if in Formula (1), we change n to $(n - 1)$ and subtract, we find,

$$\sum_{o}^{n} (bh) - \sum_{o}^{n-1} (bh) = z_n - nd_n + (n-1)\ d_{n-1}$$
$$= (n-1)\ (d_{n-1} - d_n) + (z_n - d_n) \ldots\ldots (4)$$

This formula can also be obtained directly from the figure. Thus, take $n = 4$, $n - 1 = 3$, then the left number of Formula (4) equals $3\ (b_4\ b_3) + b_4\ h_4$, which reduces to $3\ (d_3 - d_4) + (z_4 - d_4)$, or to the right member.

Similarly, in Formula (2), change n to $(n - 1)$ and subtract. Therefore,

$$\sum_{o}^{n} (bh.z) - \sum_{o}^{n-1} (bh.z) = z_n^{\,2} + d_{n-1} \sum_{o}^{n-1} (z) - d_n \sum_{o}^{n} (z)$$
$$= z_n^{\,2} + d_{n-1} \sum_{o}^{n-1} (z) - d_n \left(\sum_{o}^{n-1} (z) + z_n \right)$$
$$= (d_{n-1} - d_n) \sum_{o}^{n-1} (z) + (z_n - d_n)\ z_n \ldots\ldots\ldots (5)$$

A similar procedure, using Formula (3), gives,

$$\sum_{o}^{n} (bh.y) - \sum_{o}^{n-1} (bh.y) = z_n\, y_n - d_n \sum_{o}^{n} (y) - d_{n-1} \sum_{o}^{n-1} (y)$$
$$= (d_{n-1} - d_n) \sum_{o}^{n-1} (y) + (z_n - d_n)\ y_n \ldots\ldots\ldots (6)$$

Formulas (5) and (6) can likewise be derived directly from the figure by developing the left members and reducing. It will be observed that Formulas (4), (5), and (6), are equivalent to the author's formulas on noting that $d_n = \frac{L}{2} - p_n$.

To illustrate the application of Formulas (1), (2), and (3), take the segmental arch considered in the writer's "Theory of Solid and Braced Elastic Arches,"* where only eight divisions of the semi-arch were made. Of course, in a practical design, a greater number of divisions are essential for fairly accurate results, so that this particular investigation is only intended to illustrate the method for any arch, using any number of dimensions.

* Chapter V, Second Edition.

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The neutral line of the arch is shown in Fig. 3, also the two sides of the equilibrium polygon for each load P_n ($n = 1, 2, 3, \ldots$) are drawn from the computations in the book.

To effect these computations, the sums, $\Sigma\,(bh)$, $\Sigma\,(bh.z)$, $\Sigma\,(bh.y)$, are needed, and these will now be found for each load by aid of the writer's Formulas (1), (2), and (3), which will thus show the great saving in the labor of computation over the method used in the text.

The values of the vertical ordinates, $y_1, y_2, \ldots$, at the points, $a_1, a_2, \ldots$, the horizontal distances, $z_1, z_2, \ldots$, of these same points from the center of the span, and the horizontal distances, $d_1, d_2, \ldots$, of the unit loads, $P_1, P_2, \ldots$, from the center of the span, were all measured from a large-scale drawing, and their numerical values are all inserted in Tables 2, 3, and 4, together with certain derived sums needed in the application of the formulas. The method of computation is sufficiently indicated in Tables 2 and 3. In Columns 16, 18, and 20 of Table 3 are found $\Sigma\,(bh)$, $\Sigma\,(bh.z)$, $\Sigma\,(bh.y)$, for the trial equilibrium polygons corresponding to each load, $P_1, P_2, \ldots, P_8$ in turn. From these derived quantities, by use of very simple formulas given in the text quoted, the values of the horizontal thrust, the vertical components of the reactions at the springings, and the ordinates of the equilibrium polygons at the loads and at the springings are quickly found for each load, P. The equilibrium polygons are now to be drawn, as shown in Fig. 3, and the arch investigated for the actual live and dead loads.

In Table 4, the quantities, $\Sigma\,(bh)$, $\Sigma\,(bh.z)$, $\Sigma\,(bh.y)$, are computed by the author's method of differences, using Formulas (4), (5), and (6). Some care must be exercised here to avoid mistake. Thus, suppose the quantities pertaining to P_3 are to be computed. Substitute $n = 3$ in each of the formulas at the tops of the columns and put the results in the same line with P_3. Then add the results in Columns (4) and (5) on the same line with P_3 to the previous $\sum_0^2 (bh) = 6.05$ to get $\sum_0^3 (bh) = 10.46$, as given in Column (6). Similarly, proceed with Columns (8) and (9) to find $\sum_0^3 (bh.z) = 117.01$, as given in Column (10), and with Columns (12) and (13) to find $\sum_0^3 (bh.y) = 40.47$, as given in Column (14).

With a little care, it is hardly necessary to repeat the numbers added, so that the table can be compressed to about one-third the space here used. As a complete check on the results, the values given for $n = 8$ in Columns (6), (10), and (14), can also be found by use of Formulas (1), (2), and (3), exactly as indicated in Table 3, where n is given the value 8. Further, a check may be given where any value of n, as $n = 4$, is reached, by the use of the same Formulas

Mr. Cain. (1), (2), and (3). The independent method illustrated in Table 3 is somewhat shorter than the difference method, but it can only be checked by repeating the computations. This complete check on the work is the principal gain afforded by the author's method of differences, and he is to be congratulated on having derived such a brief and valuable method of computation.

In conclusion, a few words may not be inappropriate concerning the advantage of the method of single loads in arch analysis. This consists not only in an increased accuracy, but it is the only practicable method by which maximum fiber stresses can be ascertained. Thus, after the equilibrium polygons for the various unit loads have been

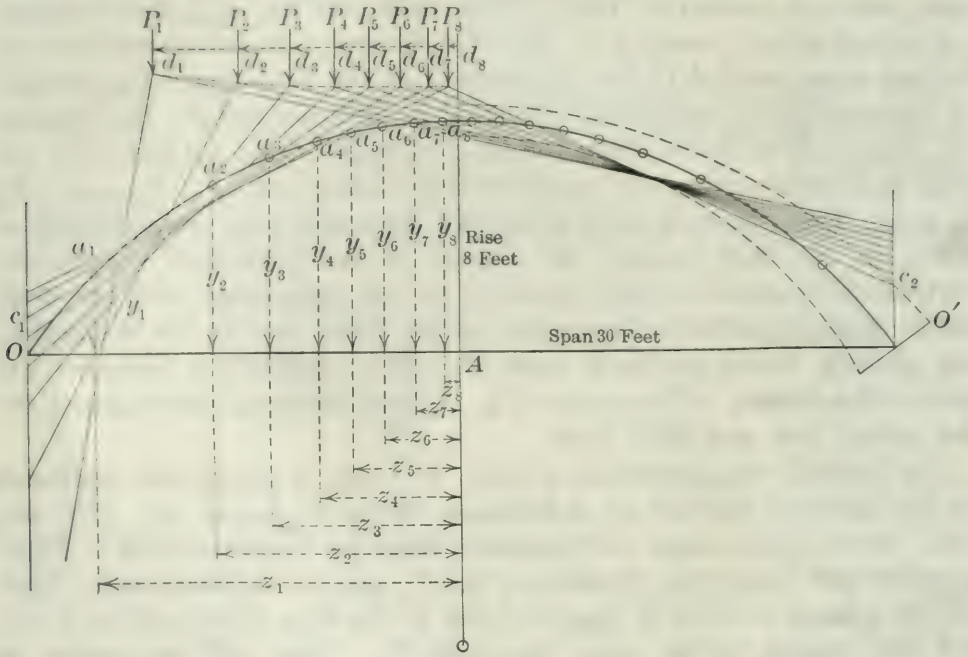

FIG. 3.

drawn, the exact position of the live load to give the maximum stress at any section of the arch can be at once determined from the figure. The writer has shown* the great variations in stress at the critical sections of an arch as a live load moves across it, and has indicated probable positions for this load to cause maximum fiber stress at certain sections. Evidently, however, the positions will depend on the shape of the arch, and will thus vary as the arch is thicker or thinner and also with the curves of the intrados and extrados. The method of single loads gives certainty, the usual method uncertainty, in endeavoring to find maximum stresses. It is feared, therefore, that of the tens of thousands of concrete arches built in the United States, the maximum stresses in but very few have been ascertained. The subject is a vital one, and of practical importance.

* "Theory of Solid and Braced Elastic Arches."

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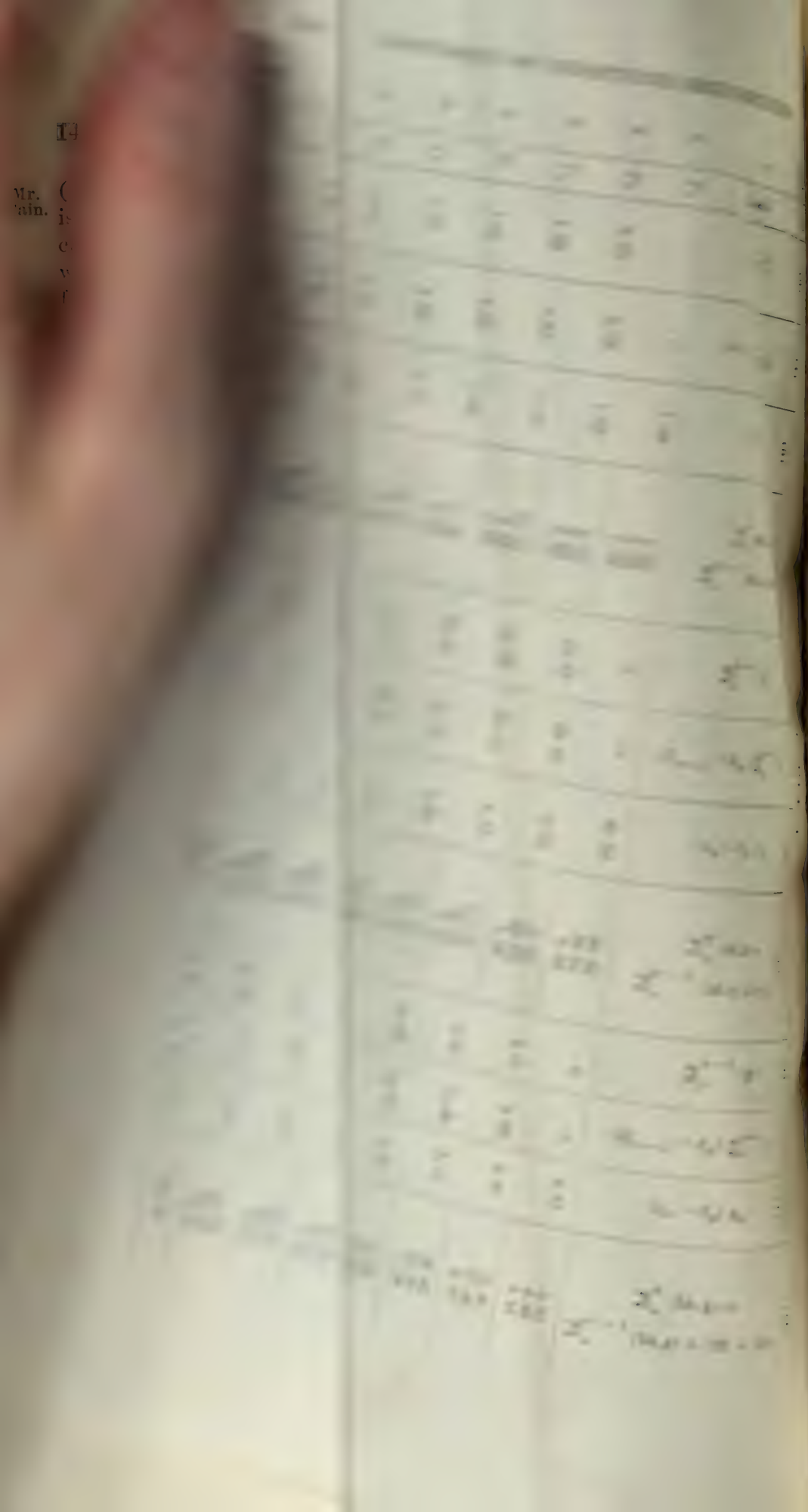
TABLE 2.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
n	Point.	y	$\Sigma_0^n (y)$	y^2	z	$\Sigma_0^n (z)$	z^2	$\Sigma_0^n (z^2)$	$z\,y$	$\Sigma_0^n (z\,y)$
1.....	a_1	2.77	2.77	7.67	12.72	12.72	161.80	161.80	35.23	35.23
2.....	a_2	5.74	8.51	32.95	8.73	21.45	76.21	238.01	50.11	85.34
3.....	a_3	6.72	15.23	45.16	6.65	28.10	44.22	282.23	44.69	130.03
4.....	a_4	7.28	22.51	53.00	5.08	33.18	25.81	308.04	36.98	167.01
5.....	a_5	7.60	30.11	57.76	3.79	36.97	14.36	322.40	28.80	195.81
6.....	a_6	7.81	37.92	61.00	2.63	39.60	6.92	329.32	20.54	216.35
7.....	a_7	7.92	45.84	62.73	1.53	41.13	2.34	331.66	12.12	228.47
8.....	a_8	7.99	53.83	63.84	0.51	41.64	0.26	331.92	4.07	232.54
		53.83		384.11	41.64		331.92			

TABLE 3.

(12)	(13)	(14)	(15)	(16) = (7) − (15)	(17)	(18) = (9) − (17)	(19)	(20) = (11) − (19)
n	Load.	d_n	$n d_n$	$\Sigma_0^n (bh) = \Sigma_0^n (z) - n\,d_n$	$d_n\ \Sigma_0^n (z)$	$\Sigma_0^n (bh.z) = \Sigma_0^n (z^2) - d_n\ \Sigma_0^n (z)$	$d_n\ \Sigma_0^n (y)$	$\Sigma_0^n (bh.y) = \Sigma_0^n (z\,y) - d_n\ \Sigma_0^n (y)$
1.....	P_1	10.73	10.73	1.99	136.49	25.31	29.72	5.51
2.....	P_2	7.70	15.40	6.05	165.16	72.85	65.53	19.81
3.....	P_3	5.88	17.64	10.46	165.23	117.00	89.55	40.48
4.....	P_4	4.43	17.72	15.46	146.99	161.05	99.72	67.29
5.....	P_5	3.22	16.10	20.87	119.04	203.36	96.95	98.86
6.....	P_6	2.09	12.54	27.06	82.76	246.56	79.25	137.10
7.....	P_7	1.02	7.14	33.99	41.95	289.71	46.76	181.71
8.....	P_8	0.24	1.92	39.72	9.99	321.93	12.92	219.62



N SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 124

REINFORCED CONCRETE JOISTS.

By JOHN L. HALL, M. Am. Soc. C. E.

WITH DISCUSSION BY MESSRS. ERNEST McCULLOUGH AND JOHN L. HALL.

In computing the strength of reinforced concrete floor slabs, it is usual to neglect the tensile resistance of the concrete. That portion of the concrete on the tensile side of the neutral plane is considered necessary for covering the steel, for bonding to resist shear, and for fireproofing. A small part of this concrete would ordinarily be sufficient to cover the steel and furnish the necessary resistance to shear. The remainder is a heavy and somewhat expensive material for floors. Particularly is this true in the case of long spans which require thick slabs.

By omitting the reinforcing steel in large units, a series of parallel ribs or joists may be formed, instead of a flat slab. With a thin slab at the top, lightly reinforced transversely, the joists become a series of small T-beams. The expensive form work of such a system is the objection to it, and the preference for a flat ceiling is another. To meet these objections, burned hollow tile with plaster ceiling, or sheet-metal tile with mortar filling and plaster ceiling, has been used.

The purpose of this paper is not to discuss the relative merits of these several methods of construction, but rather to consider the things which should be considered in computing the strength of such a system of joists. Various claims are made as to the strength of floors formed by clay tile in combination with concrete joists. In

TABLE 4.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
n	Load.	$d_{n-1}-d_n$	$(n-1)(d_{n-1}-d_n)$	z_n-d_n	$\Sigma_o^n(bh)=\Sigma_o^{n-1}(bh)+(4)+(5)$	$\Sigma_o^{n-1}(z)$	$(d_{n-1}-d_n)\Sigma_o^{n-1}(z)$	$(z_n-d_n)z_n$	$\Sigma_o^n(bh.z)=\Sigma_o^{n-1}(bh.z)+(8)+(9)$	$\Sigma_o^{n-1}(y)$	$(d_{n-1}-d_n)\Sigma_o^{n-1}(y)$	$(z_n-d_n)y_n$	$\Sigma_o^n(bh.y)=\Sigma_o^{n-1}(bh.y)+(12)+(13)$
1...	P_1		0	1.99	1.99 3.03 1.03	0	0	25.32	25.32 38.54 8.99	0	0	5.51	5.51 8.39 5.91
2...	P_2	3.03	3.03	1.03	6.05 3.64 0.77	12.72	38.54	8.99	72.85 39.04 5.12	2.77	8.39	5.91	19.81 15.49 5.17
3...	P_3	1.82	3.64	0.77	10.46 4.35 0.65	21.45	39.04	5.12	117.01 40.74 3.30	8.51	15.49	5.17	40.47 22.08 4.73
4...	P_4	1.45	4.35	0.65	15.46 4.84 0.57	28.10	40.74	3.30	161.05 40.15 2.16	15.23	22.08	4.73	67.28 27.23 4.33
5...	P_5	1.21	4.84	0.57	20.87 5.65 0.54	33.18	40.15	2.16	203.36 41.77 1.42	22.51	27.23	4.33	98.84 34.01 4.22
6...	P_6	1.13	5.65	0.54	27.06 6.42 0.51	36.97	41.77	1.42	246.55 42.37 0.78	30.11	34.01	4.22	137.07 40.57 4.04
7...	P_7	1.07	6.42	0.51	33.99 5.46 0.27	39.60	42.37	0.78	289.70 32.08 0.14	37.92	40.57	4.04	181.68 35.75 2.16
8...	P_8	0.78	5.46	0.27	39.72	41.13	32.08	0.14	321.92	45.84	35.75	2.16	219.59

Mr. Cain.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1245

THEORY OF REINFORCED CONCRETE JOISTS.

By John L. Hall, M. Am. Soc. C. E.

With Discussion by Messrs. Ernest McCullough and John L. Hall.

In computing the strength of reinforced concrete floor slabs, it is usual to disregard the tensile resistance of the concrete. That portion of the concrete on the tensile side of the neutral plane is considered only useful for covering the steel, for helping to resist shear, and for forming a ceiling. A small part of this concrete would ordinarily be sufficient to cover the steel and furnish the necessary resistance to shear. The remainder is a heavy and somewhat expensive material for a ceiling. Particularly is this true in the case of long spans which require thick slabs.

By keeping the reinforcing steel in large units, a series of parallel concrete joists may be formed, instead of a flat slab. With a thin slab over the top, lightly reinforced transversely, the joists become a system of small T-beams. The expensive form work of such a system is one objection to it, and the preference for a flat ceiling is another. To obviate these objections, burned clay hollow tile with plaster ceiling, or sheet-metal tile with metal lath and plaster ceiling, has been used.

The purpose of this paper is not to discuss the relative merit or economy of these several methods of construction, but rather to discuss the things which should be considered in computing the strength of such a system of joists. Various claims are made as to the work performed by clay tile in combination with concrete joists. It is not

the intention to discuss this matter at present. The spaces between joists, therefore, will be assumed to be voids. Fig. 1 shows how such a system was used in a recent design. See also the plan, Fig. 6. Fig. 2 is a section through the joists, and Fig. 3 is a section through the beams. Fig. 4, illustrating the mode of bending, in a beam with fixed ends, is introduced for the purpose of reference in what follows.

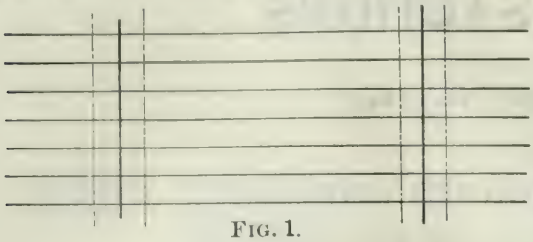


FIG. 1.

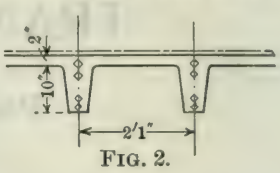


FIG. 2.

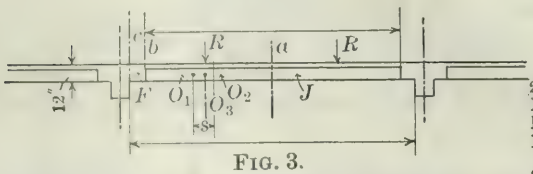


FIG. 3.

21′8″ Span
Moment at a (Fig. 3) = 4540 ft.-lb.
Reaction R = 1640 lb.
Moment at b = −7900 ft.-lb.
" " c = −11000 " "
23′10″ Span
Moment at a = 5400 ft.-lb.
Reaction R = 1790 lbs.
Moment at b = −9100 ft.-lb.
" " c = −13700 " "

Round Rods are indicated thus (ϕ)
Square, Cold-twisted Bars are indicated thus (◆)

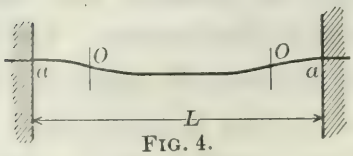


FIG. 4.

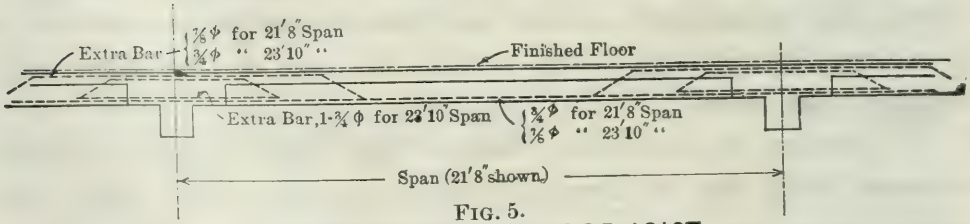


FIG. 5.
TYPICAL DETAIL OF FLOOR JOIST

The bending moment of a simple beam resting freely on end supports is determined solely by the amount and distribution of the load (including the weight of the beam itself), entirely regardless of its sectional shape or materials.

In a continuous beam, however, the bending moment depends on the amount and distribution of loads and also on the elastic curve or deflection of the beam. The elastic curve is influenced by the shape and composition of the beam. What the actual bending moments are in reinforced concrete, therefore, is not known. It is a matter of much difference of opinion among engineers, as shown

by the discussion before this Society following the Progress Report of the Special Committee on Concrete and Reinforced Concrete.

As commonly given in books on mechanics, the bending moments and reactions for continuous beams are calculated for the ideal case of homogeneous beams of uniform sections resting freely on level supports evenly spaced. Such cases do not occur in building construction. After many tests of actual construction, the formulas given in the building laws, although based on the ideal case, are recognized

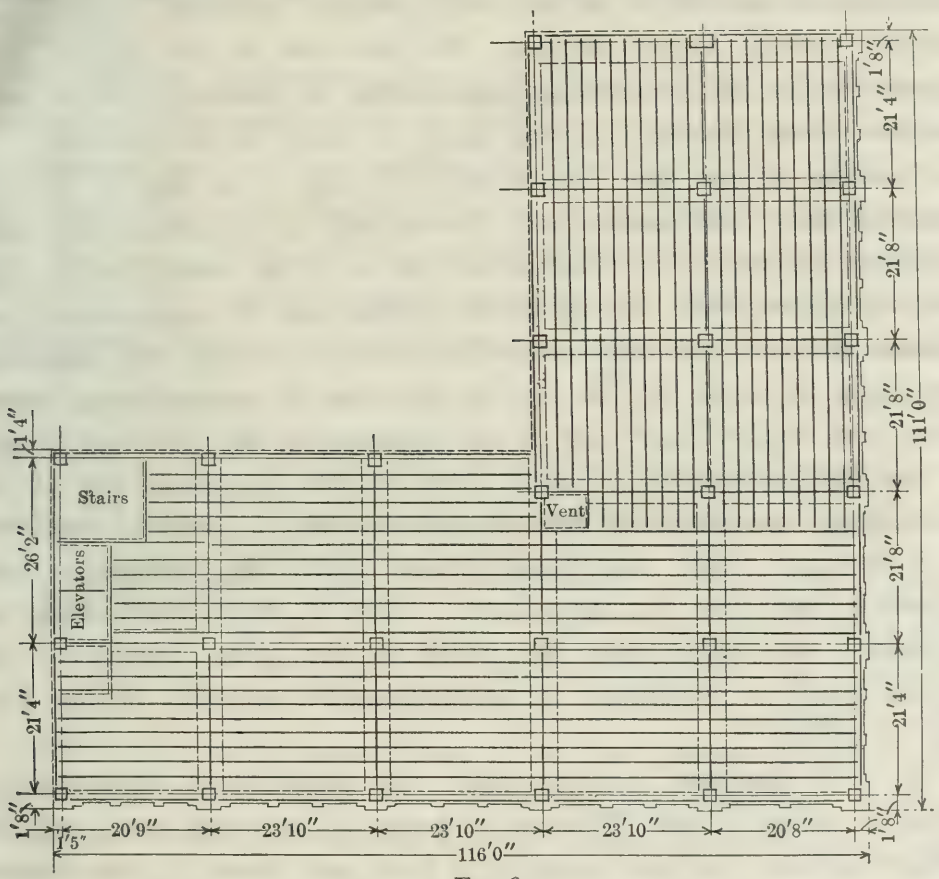


FIG. 6.
TYPICAL FLOOR FRAMING PLAN

as safe, and are in general use; yet it is well to remember always that these formulas are only approximations. The nearer a design approaches the ideal case referred to, the more nearly do the formulas approach the truth. Any material deviation in design from the ordinary approximation to the ideal condition presents a case for special study and determination.

It is thought that the laws of deflection and the amount of deflection under working loads are much the same in reinforced and

in plain concrete. Certainly, the small deflections in reinforced concrete, as compared with those in structural steel, show that in the former the concrete is more of a controlling factor in deflections than the steel. This theory is consistent also with calculated deflections using the moment of inertia of sections containing usual percentages of steel. It would seem, therefore, that the reinforcing steel has very little influence on the elastic curve of the kind of construction under investigation, except in so far as it prevents tensional rupture and thereby permits higher stresses. In this view of the subject it is apparent that the reinforcing steel should be placed so as to resist rupture where it would be most likely to occur in the concrete.

Looking at the floor construction of Figs. 1, 2, and 3, if it be assumed that this construction is essentially a continuous flat slab, and that the maximum bending moment is at the center line of supporting beams, then the maximum stress both in compression and tension will be along this center line, and the stresses will diminish, according to some law, to zero at the line of contraflexure. Suppose, now, that a large part of the concrete on the compression side of the slab be removed at some place between the center line of the supporting beam and the line of contraflexure; a plane of weakness is introduced which may cause failure where the moment is considerably less than the maximum. This is inconsistent with the premises, and shows that the width of the 12-in. flanges of the beams must be considered in calculating the strength of the floor slab so-called.

If it is assumed, however, that the construction consists of a series of joists or small T-beams with one or both ends fixed, then the danger section of a joist occurs at a fixed end, where it joins a beam, and the analysis becomes straightforward and consistent. It is thought, therefore, that the joists should be designed for fixed ends, in accordance with actual conditions.

An objection which might be made to this procedure is that the beam flanges, into which the ends of the joists are said to be fixed, are themselves capable of deflection, so that the ends of the joists are inclined slightly instead of level. This deflection at the edge of the flanges, however, must necessarily be very small, being estimated according to the respective moments of inertia, and is only 40%

of what it would be at the same line if the joist section ran without change to the center of the beams.

The effect of this slight inclination of the supports would be to increase slightly the positive moment at the center of the joists and to reduce slightly the negative moment at the fixed ends. This reduction of negative moment would cause the calculated negative moment to err on the side of safety, and the slightly increased positive center moment would utilize more economically the excess strength provided at the center.

The negative bending moment along the center line of the beams is the sum of the moments of the distributed loads out to the line of contraflexure, and of the concentrated loads along the latter line. Any change in design that tends to move the line of contraflexure away from the supports and toward the center of the span, would tend to increase the negative moment at the center of the supports. It is conceivable, therefore, that the negative moment in the case under discussion might be somewhat more than $\frac{WL}{12}$. If so, additional steel should be provided in the top of the slab across the beams.

The formula, $\frac{WL}{12}$, expresses the bending moment at either end of a beam of constant section uniformly loaded and having fixed ends. The points of contraflexure, *O* (Fig. 4), are located $0.211L$ from the ends. The central portion, *O-O*, may be considered as a simple beam, uniformly loaded. The end portion, *a-O*, may be considered as a cantilever uniformly loaded from *a* to *O* and supporting at *O* the reaction from *O-O*. From these conditions the shear and bending moment can be readily computed for any section in the length of the beam.

In similar manner, a continuous beam is restrained by bending moments at the supports. The points of contraflexure, however, from which the moments may be computed, are not located as easily. Their position is affected by the number and the relative length of the spans and by the distribution of the live loads, whether on some or all of the spans, and by other considerations. In order to simplify the computations, the building laws authorize the use of the formula, $\frac{WL}{12}$, for interior spans, and, $\frac{WL}{10}$, for end spans. This method of

calculating is only approximately correct. Its error is usually, although not always, on the side of safety. The formula for interior spans is the same as that for beams with both ends fixed. For end spans, the formula indicates one fixed end and one partly fixed.

When applicable, the theorem of three moments permits the accurate determination of moments and shears for actual conditions; and, if all the conditions are actually considered, it affords a more scientific method of calculation than the approximate formulas previously stated. The building law recognizes the validity of scientific analysis, and caution would seem to require such analysis, if attainable, whenever the design varies materially from ordinary conditions of continuity, as, for example, when the spans are very unequal in length, or when the conditions of constant section and free support are deviated from in any marked degree.

The features of this design, which vary from usual conditions of continuity, are: (1) the massive character of the supporting beams; and (2) the sudden change in section where the joists join the flanges of the beams.

The joists in interior spans were assumed to have fixed ends, and an attempt was made to determine the location of the points of contraflexure. Investigations by F. E. Turneaure, Assoc. M. Am. Soc. C. E., indicate that, within ordinary working stresses and percentages of steel, the elastic curve of a concrete beam is not greatly influenced by the position of the reinforcement. It appears to be desirable, therefore, to ascertain the natural elastic curve of the concrete joists and place the steel where it is needed in conformity thereto.

Two assumptions were tested: (1) That the ends of the joists are fixed at the face of the beam web; (2) that they are fixed at the edge of the beam flanges. The location of the points, O, was calculated for both conditions, on the basis of a constant section.

The position, O_1, Fig. 3, obtained by the first calculation, would be correct if the flange, F, were as flexible as the joist, J, while the position, O_2, would be correct if the flange, F, were perfectly rigid. As the flexibility of F is intermediate between these assumptions, the true position, O_3, must lie between O_1 and O_2, and closer to the one derived from that assumption, which is nearer the truth.

A beam of the section, F, it was estimated, would deflect four-tenths as much under a given load and span as one of the section J

It is then more nearly a rigid beam than one of the same flexibility as J. The point, O_3, therefore, lies nearer O_2 and is four-tenths of the way from O_2 toward O_1.

Having fixed the position of the points, O, the remaining calculations are very simple. The typical detail of the joist, Fig. 5, is designed in accordance with the actual conditions as understood, and the necessary resistance is provided. The moments and shears are stated on that figure.

In view of the fact that the moment at the center of the span of a beam with fixed ends is only one-half of $\frac{WL}{12}$, it may be asked why this formula is required by law generally for the center of continuous interior spans, and whether such requirement would be justified in the present instance. Analysis of the spans under consideration by the ordinary theory of continuous beams showed that if only one span be fully loaded, the other spans having dead load only, the fully loaded span will have a moment at the center about two-thirds of $\frac{WL}{12}$. If the live load on one span only were greatly increased over the dead load, the moment at the center of the span would be increased in a larger ratio; but, in the absence of those conditions, there appears to be nothing gained by increasing the reinforcement at the center of the span.

Again, the theory of continuous beams does not take into account the torsional stiffness of the supporting beams, which, in the present case, is very considerable. Manifestly, the more nearly we approach the ideal condition of fixed end supports for the joists, the less influence will be exerted by conditions outside the particular span considered, so that the probable maximum moment at the center of the span of the joists will always be less than two-thirds of $\frac{WL}{12}$, in this particular design.

In the detail herewith submitted, Fig. 5, the reinforcing steel is disposed in accordance with the foregoing statement of theory.

DISCUSSION

Mr. McCullough.

ERNEST MCCULLOUGH, M. AM. SOC. C. E. (by letter).—In some recent correspondence with Sanford E. Thompson, M. Am. Soc. C. E., the writer's attention was called to a too common error on the part of designers in reinforced concrete, wherein the sum of the middle bending coefficient and the coefficient at one support is always assumed to be equal to $\frac{1}{8} wl^2$. The moment of inertia of the section is not considered as being affected by the quantity of steel, the result being that, if a middle coefficient $= \frac{1}{10}$ is assumed, the coefficient over supports is taken as $\frac{1}{40}$. The writer checks a great many plans for contractors and owners, these plans being generally furnished free of cost by steel salesmen. The almost uniform practice is to use the clear span, face to face of T-beams, for floor slabs, and the clear span, face to face of columns, for beams and girders, the foregoing coefficients being used for steel distribution. The writer has never been able to make the designers do more than use a coefficient of $\frac{1}{12}$ in the middle and $\frac{1}{24}$ over supports. He has sometimes failed to secure a concession of even that much, as the designers can refer owners to a large number of buildings, erected in every State in the Union, designed with the foregoing distribution of bending coefficients. When possible, he compels the use of $\frac{1}{12}$ over supports and $\frac{1}{18}$ in the middle, whereas, the building ordinances of Chicago and most cities call for a total of $\frac{1}{6} wl^2$, with a coefficient of $\frac{1}{12} wl^2$ in the middle, of course, then obtaining equal steel areas in the middle and over each support.

Two weeks ago a designer gave as authority for his method, Turneaure and Maurer's "Principles of Reinforced Concrete Construction," second edition, page 307, where the following occurs:

"* * * but the use of $\frac{1}{10}$ for the general coefficient will provide ample strength in all ordinary cases. This would require an actual resisting moment at each support of only about $\frac{1}{40} pl^2$ in order that the center moment be reduced to $\frac{1}{10} pl^2$."

His attention was called to a line on the same page where the following appeared:

Mr. McCullough.

"The foregoing calculations assume uniform moment of inertia and therefore that about the same amount of steel is used for negative as for positive moments. The effect of variation in moment of inertia is discussed in Art. 166."

In the preceding paragraph, on the same page, the authors propose certain coefficients, saying plainly that they are for both positive and negative moments. To the writer, it seems impossible to make any mistake in the matter, if it is carefully studied, even with no other authority than that above quoted.

Turning to Art. 166 in the above-mentioned book, it is found to deal with columns, the reference being a misprint, Art. 165 being meant. This section is a valuable discussion of the effect of varying moments of inertia, being based, for the most part, on a discussion by P. E. Stevens, Assoc. M. Am. Soc. C. E.* A table is given, based on the moments of inertia varying with the quantities of steel, showing that if the assumption be made that the middle coefficient $= \frac{1}{8} wl^2$ and about one-fifth of the quantity of steel in the middle in the bottom be placed in the top over supports, the stress at the center will be 55% of the working stress and at the end will be $4.1 \times 0.55 = 2.25$ times the working stress.

In the last edition of "Concrete, Plain and Reinforced," by Taylor and Thompson, the following is found on page 439:

"In applying this to the various cases, the assumption is made that the moment of inertia of the beam is constant throughout its length. While this is not strictly true, extensive studies of various cases in reinforced concrete show that a large change in the moment of inertia makes a very small change in the bending moment, so that the relations are substantially correct until a member enters a much larger member."

Mr. Thompson, in a letter to the writer, says:

"Many designers have the very erroneous idea that, because they design a beam for $\frac{wl^2}{8}$, the bending moment is reduced to zero over the supports. This, of course, is not the case, because designing the beam for $\frac{wl^2}{8}$ is an entirely different matter from changing the bending moment to $\frac{wl^2}{8}$. The bending moment still remains in the neighborhood of $\frac{wl^2}{24}$, if the beam is fully continuous; so that the bending moment at the support for uniform load is still a little less than $\frac{wl^2}{12}$."

* *Transactions*, Am. Soc. C. E., Vol. LX, 1908, p. 496.

Mr. McCullough.

In his paper, Mr. Hall has placed the matter in the proper light, but has neglected to call attention specially to the fact that all concrete designs, wherein continuity of construction is considered, should be based on the end moments rather than the center span moments.

A few days ago, in conversation with three structural engineers, members of this Society, the writer discovered, to his surprise, that they believed the sum of the center and one end coefficient $= \frac{1}{8}$; and if the assumption is made for a large bending moment in the center, that only enough steel is required over supports to care for the remainder of the coefficient, it being their practice to use arbitrarily over supports about one-fifth of the quantity of steel used in the middle, in the event that they use $\frac{wl^2}{8}$ in the center. This small quantity of steel they claim is to take care of possible cracks, mainly caused by temperature, and possibly by a bending down of the flanges of T-beams, or unlooked for variations in loading. As such men hold to these views, it is proper to have a full discussion of this phase of the subject, and that some present methods of designing be discouraged. Five or six years ago such views were common, and it is surprising that by this time so many men of standing in concrete work still neglect consideration of the fact that "designing the beam for $\frac{wl^2}{8}$ is an entirely different matter from changing the bending moment to $\frac{wl^2}{8}$," as stated by Mr. Thompson.

Mr. Hall.

JOHN L. HALL, M. AM. SOC. C. E. (by letter).—The writer has read with interest the discussion by Mr. McCullough, who seems to accept the theory. His discussion emphasizes the fact that the prevailing practice among designers of continuous reinforced concrete construction errs in assuming the position of maximum bending moment at the middle of the span, instead of at the supports. Judging from the building regulations of the principal cities of the United States, which the writer has had occasion to examine during the past two or three years, it would seem that Mr. McCullough has much evidence to justify his contention. It is thought to be good practice to calculate the moments as nearly as may be in accordance with the actual conditions of loading, and on the ordinary theory of mechanics.

The records of tests of reinforced concrete and the literature of the subject leave some doubt in the mind as to what the actual bending moments are at the middle of the span in reinforced concrete structures, but it is thought that the moments at the supports are in general as large as the results obtained by the methods proposed by the writer.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1246

PORTS OF THE PACIFIC.*

By H. M. Chittenden, M. Am. Soc. C. E.,
Assisted by A. O. Powell, M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Le Conte, E. P. Goodrich, Lewis M. Haupt, and H. M. Chittenden.

To describe any one of the larger ports of the Pacific Coast adequately would exceed the admissible limits of this paper. To describe them all with anything like minute detail is clearly impossible, and it is assumed that, in assigning this topic, the intention was that it be treated only on broad general lines which should give a comprehensive view of the subject as a whole, rather than a study in detail. Even with this understanding, the scope of the subject is so vast that its compression into the compass of a professional paper seems little better than the preparation of a table of contents. The attempt has been made, however, and its results are presented herewith under the following headings:

I.—Strategic Relations of Coast Ports.
II.—Descriptive Data.
III.—Engineering Problems.
IV.—Administrative Systems.
V.—Plans for the Future.
VI.—Influence of the Panama Canal.

The paper will treat only of those ports north of the boundary between Mexico and the United States.

*This paper was presented to the Annual Convention at Seattle, Wash., and also at the meeting of November 20th, 1912.

I.—Strategic Relations of Coast Ports.

Commerce of the Pacific.—Since the days of Magellan, imaginative minds have pictured the Pacific Ocean as the future home of the world's commerce. There is something in the immensity of that ocean, in the present vastness of the population on one shore and the future vastness of that on the other, which conjures up visions of argosies such as the Mediterranean or the Atlantic has never known. The situation as it actually exists, however, makes this picture somewhat of an illusion. The very wideness of the intervening sea is a mighty barrier to economic intercourse across it. The breadth of the Pacific as compared with that of the Atlantic, depending on the latitude, is as two or three to one; and, while the cost of transit and loss of time may not be in the same proportion, there is still a wide disparity against the larger ocean. There are also the almost insuperable barriers of race and economic conditions on the Asiatic Coast. We do not admit the Oriental to our shores as we do the European. That immense source of traffic which has sustained its thousands of ships on the Atlantic is cut off on the Pacific. Likewise those Oriental countries have little to offer the traveler which can compare with the lure of European civilization, ancient and modern. Tourist traffic across the Pacific is a bagatelle compared to that over the Atlantic. Finally, the deep poverty of the hordes who swarm the Asiatic shores, and the backward condition of industry there, are not promotive of vigorous commercial intercourse, for they offer relatively little to sell and less with which to buy.

Thus it results that the commerce of this ocean, great though it be, is small compared with that of the Atlantic, and small to what might be expected from the millions of inhabitants affected by it. Its growth in the near future, strange as it may appear, is more dependent on the countries which border the Atlantic than on those washed by its own waters. As far as the Pacific Coast is concerned, the eyes of its people are turned east rather than west. It is there that their kindred dwell; it is there that the capital exists which shall develop their boundless resources; it is thence that must come those who shall help to populate their shores. It is this fact—that their future is on the Atlantic rather than on the Pacific—which makes the opening of the Panama Canal an event of such tremendous import to them. It is bringing them to their own. It fronts them toward

Europe. It opens to them the treasures of the Occident—far more potent, if less romantic, than those of the Orient. It gives them something of the advantage which the eastern shore of the Continent enjoys by reason of its closer relation to the fundamental sources of our civilization.

If this condition is true of the present, and will remain so, to a degree, for the indefinite future, let it not blind us to the profound changes which are taking place in those ancient, and, but yesterday, non-progressive, countries which lie on the other side of the Pacific. When we consider what a volume of commerce the accidents of political fortune have turned in this direction from the distant Philippines; when we reflect on the marvelous progress of modern Japan; and when we note the amazing changes now going on in that venerable nation which has slumbered indifferent to the rest of the world for thirty centuries or more, we cannot afford to take a pessimistic view of our future relations with that side of the globe. If ever these people accomplish what is within their practical reach; if ever they turn to account, as is done in Europe and America, their wealth of natural resources and their own capacity for industrial development, surely there will result a mighty increase in trade between them and the rest of the world, and of this the American Continent will enjoy the greater share.

Thus, while the Atlantic holds out to us the brighter present prospect, there is brilliant promise in the Pacific, and in every direction the future is big with hope. Visions of the coming day are profoundly stirring the minds of men. Expansion is in the air. The measureless force of unseen psychological influences rushes the world along whether it will or no. Doubtless, it builds exaggerated hopes and paves the way to much disappointment, but its very exuberance of faith is an earnest of vast accomplishment.

This far-reaching movement, which has been crystallized into definite form by the approaching consummation of the greatest engineering work of ages, finds its intensest expression along the Pacific Coast of North America. To other parts of the world, the Panama Canal means simply increased opportunities for trade; to the Pacific Coast it means a new lease of life through the elimination of those barriers which separate it from its true source of sustenance and growth. Everywhere along the Coast, faith in the beneficial results

of this great work is unbounded. It is a faith, moreover, which expresses itself in works. From Alaska to Lower California, the Coast is getting ready for the Canal. It is putting its house in order. It is spending in this work prodigious sums of money. The present decade will witness an expenditure in port development of probably $50 000 000. The slogan which has won this vast sum from the pockets of the taxpayers is: "Get ready for the opening of the Panama Canal," and the formal celebration of that event will find the work well toward completion.

It would be idle to pretend that this prodigious effort springs solely from an actual necessity of providing for the increase of traffic that will result from the opening of the Canal. The popular belief is, of course, that this is the case; but those who have studied the situation closely know better. They realize that the movement is being overdone, but they recognize that it is bound to keep on through the fear which each port entertains of what its sister ports may do. The stigma of possible failure in the race and fear of loss of prestige are the potent forces which are back of these extraordinary efforts. Los Angeles or Seattle would find it difficult on the cold basis of rational business foresight to justify their enormous prospective outlays; but they find ample justification in the necessity of keeping up with a procession which now stalks with tremendous strides from one end of the Coast to the other.

Competition of Ports.—This interesting state of things suggests a primary basis for comparing the ports of this Coast, namely, their relations to one another as commercial rivals. In the long stretch of coast line from Latitude 32° 30′ to Latitude 54° 40′, there are certain points of true strategic value; that is, points the possession of which gives the possessor important advantages over his competitor. Broadly speaking, there are two great divisions of the Coast from a commercial point of view—the north and south—with the Oregon-California line the approximate boundary between them. Only in a very general sense are these two sections of the coast competitors. They both reach out, it is true, to Hawaii, the Orient, and Alaska, and inland to common interior points; but each has some advantage peculiar to itself—a sort of proprietary right in certain spheres in which the other seems rather an interloper. The competition in such cases

is not keen, for the advantage of the more favored rival is too pronounced to be coped with successfully.

It is in a narrower sphere that the competition of these ports becomes really intense. San Francisco, Los Angeles, and San Diego have little to fear from the ports of the North, for their respective hinterlands merge only in the far interior of the Great Basin, but they have a wholesome respect for one another. Portland poaches but slightly on San Francisco's preserves, and Puget Sound still less, but the port on the Columbia never for an instant lets her sisters farther north forget her existence. Puget Sound and the ports of British Columbia are also competitors of a strenuous type in spite of the artificial barriers of nationality, tariff, and different customs and laws. In these local groups competition is really keen, and each competitor is feverishly anxious as to what its rivals are doing, while the still smaller units of the group flourish either by virtue of special advantages or simply on the crumbs which fall from their wealthier sisters' tables.

San Francisco Bay.—Having thus touched in most general terms on the trade relations of different sections of the Pacific Coast, let us consider in some detail the principal ports, still from the point of view of their commercial relations. Whatever changes the future may have in store, it is now true, and for a long while will so remain, that San Francisco Bay is far and away the most important port on the Coast. It is a wonderful port—wonderful in the strategic relation to its California hinterland and to the great interior of the country; wonderful in its physical conformation as a vast sheltered harbor opening in, through a narrow and easily defended entrance, from a coast line almost devoid of harbors for hundreds of miles in either direction; wonderful in its romantic history, and wonderful in its relation to the commerce of the world. Nature wrought a masterpiece when she made San Francisco Bay. Its great expanse and its navigable connections north and south, through the rich valleys of the San Joaquin and Sacramento, fit it perfectly as the entrepôt of a vast empire. The work of Nature was supplemented by the good offices of fortune, which early turned the attention of the world to this port and laid the foundation of its future greatness so deep that neither earthquake nor the growth of rivals can shake it. The Golden Gate—named three centuries before, in beautiful prophecy of

the Argonauts of Forty-nine, whose anchors dropped into yellow sands brought down by the slicken-laden streams of the Sierra—was the scene of a mighty commerce while yet only random traders sought the furry wealth of the harbors farther north. The first transcontinental railway had poured its traffic into the valley of the Sacramento for twenty years before any other portion of the Coast was similarly favored. San Francisco had written the most important chapter of her history while her sister ports were still almost unknown to the world. Congress did well when it selected the California metropolis as the site for the celebration of the opening of the great Canal. What a contrast it will be—the struggling mass of humanity and freight on its way across the fever-stricken Isthmus to the land of golden promise in Forty-nine, and the floating palaces which will then pass safely through Culebra Hill to a scene of resplendent riches undreamed of by even the wildest imagination of sixty-six years before! Where else on the round earth has modern progress wrought so great a change in so short a time?

Los Angeles and San Diego.—In Southern California, the location of highest strategic value, from a commercial point of view, unfortunately, happened to be where Nature did not provide a harbor, while the one really good natural harbor on that section of the Coast is not located where it will best serve the commercial needs of the country. Los Angeles, by virtue of its relation to the great Southern trade routes east, its marvelous resources in agriculture, its close proximity to the oil fields of Southern California, and the inestimable wealth of its winter climate (not exclusive in this, however), is incontestably the center of activity of Southern California. So little did Nature do in the way of providing a port in that vicinity, however, that the town did not grow up on the seashore at all, but twenty miles inland, and the possibility of making it a genuine seaport was clearly an afterthought in its development. Now, this consummation is its chief ambition, and a superb effort is being made to realize it. Los Angeles will become a great port, not because Nature made it so, but because her own virile people have said so. Its future harbor will be almost wholly artificial, but it will be a great harbor nevertheless, and will stand all the more to the credit of its people because of the sacrifices which they will have made to obtain it.

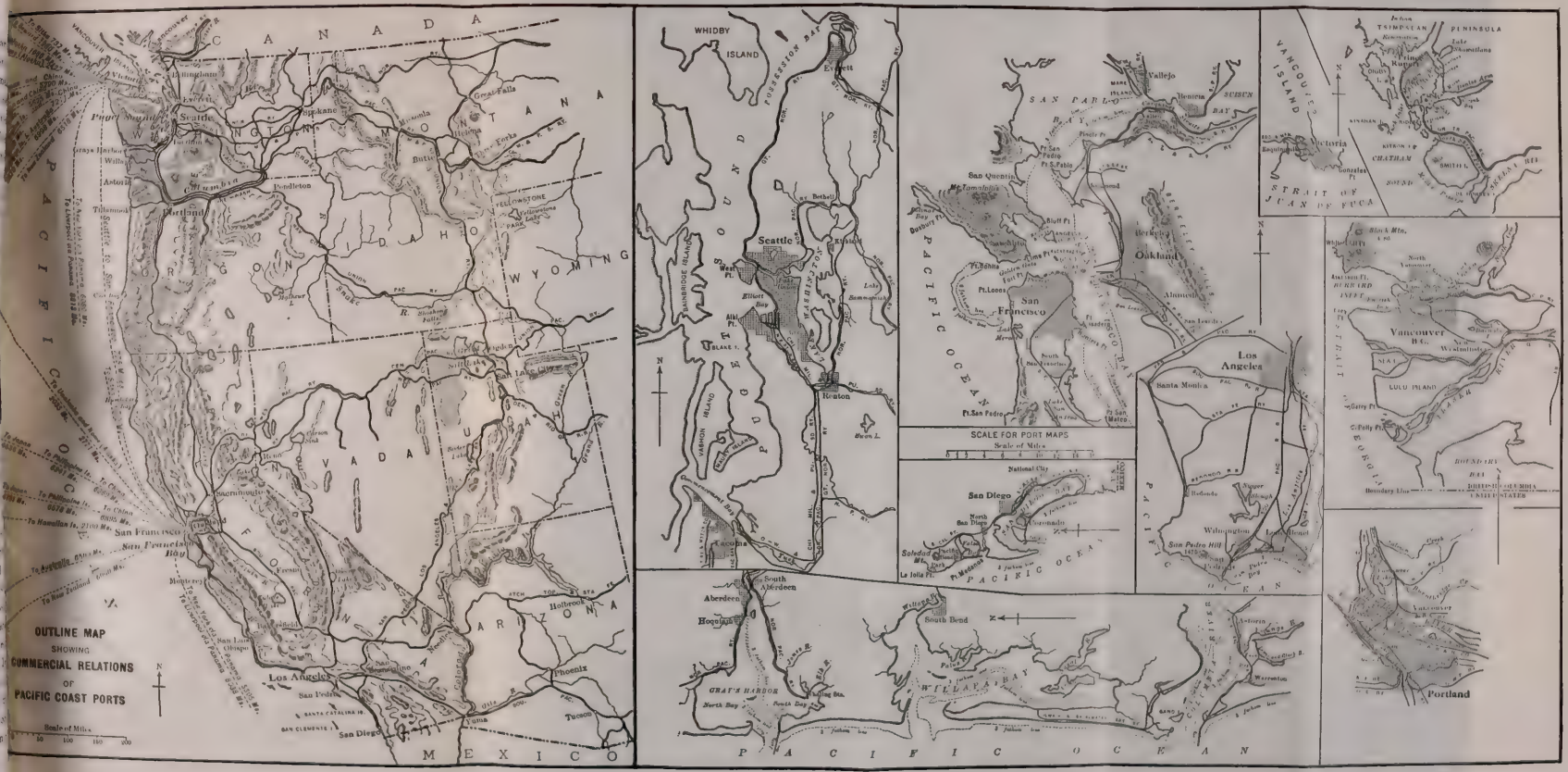
Southeasterly from Los Angeles, 89 miles, lies the splendid land-locked harbor of San Diego, ideal in its physical advantages. The fact that it never has become of first importance, however, shows that it requires something besides Nature's aid to make it a great port. It is not well located with reference to rail routes east; it has not as good a tributary country as Los Angeles; it is hemmed in by bold uplands close to the shore; and, worst of all, it suffers the purely artificial handicap of close proximity to the National frontier. Still, in spite of these drawbacks, the great natural advantages of the harbor, the lack of such advantages at Los Angeles, and the arbitrary rail tariffs which until recently have made freight from Port Los Angeles to the main city cost as much as from San Diego itself, though four times as distant, make San Diego a powerful competitor of her sister city to the north. Of course, she is bound to lose something of her relative advantage as Los Angeles develops her port, and particularly when the proposed municipal belt line connects that city with the sea and reduces freight rates to a nominal figure.

These two Southern ports have naturally cut somewhat into the trade of San Francisco, and will continue to do so. This is to be desired, as far as they may better serve the needs of the public as a whole, and beyond that point they are certain not to prevail if the communities of San Francisco Bay do their duty. California is a State of vast extent and immeasurable resources, and its hinterland is all the country to the eastward. In this illimitable field, and with the boundless ocean to the west, there will be sustenance for all, and the final balance among the ports will be adjusted on the basis of maximum efficiency of service to the public.

The Columbia.—Northward from San Francisco the first location of high strategic value is the Columbia River—the chief river of the Pacific Slope—which here breaks through the mountain barriers and opens a low-grade route to the interior of the country. It is the only point in United States territory where the great Coast-Sierra-Cascade Barrier is completely traversed by a water-grade route. The main valley extends directly back from the sea a distance of 75 miles, where its tributaries begin to spread out until they expand like an enormous fan, giving arterial highways to a water-shed of 250 000 sq. miles, with routes across the Continental Divide to the far-spreading country beyond. The main stream is susceptible of improvement for

navigation as far as into British territory, and its principal tributaries can be navigated for short distances; but the rapid fall of most of the streams, and the invariable accompaniment of rock rapids, make the liberal use of locks a necessity, and this presupposes heavy cost. In any event, it is certain that the river will be improved in the near future so that boats can ascend at all seasons as far as Central Washington, say, to Priest Rapids. This water route and the low-grade rail routes on either side, ramifying along the tributaries in all directions, give the Lower Columbia ideal communications with its hinterland, considering the exceedingly mountainous character of the country.

Plate X shows, to any trained eye, the immense advantage of the situation at the junction of the Columbia and Willamette Rivers. It is a great cross-roads. To this point vessels from the ocean, once over the bar, can safely ascend. Here, rail and river take up the route to the far interior. South is the rich Willamette Valley and Nature's land route to California. North is the Cowlitz Valley and the route to Puget Sound. The possibilities of the situation are enormous and such as will survive all competition. The chief drawback, as far as its commercial relations with the outside world are concerned, is its connection with the sea. The Columbia Bar, built up from the detritus of a vast and steep-sloped water-shed, was originally the most formidable and dangerous known to navigation, and the problem of opening and maintaining a safe and commodious channel across it is one of the most difficult in the whole range of river and harbor engineering. Only a moderate degree of success has been obtained so far, and that at heavy cost. Other plans have been suggested, such as cutting through the sand spit to an artificial harbor to be built south of the jetty, or through the isthmus on the north to the fine natural harbor of Willapa Bay; but they have never received serious attention, and the plan of deepening the bar by jetty construction will probably continue. With it must go, hand in hand, the project of deepening the river itself in its 100-mile channel from the bar to the port. The situation is something like that at Los Angeles in that Man must be relied on to make good the large deficiency of Nature. This, as the writer has elsewhere observed, the enterprising citizenship of Portland is resolved shall be done, cost what it may, to the end that there may be realized, even more than at


OUTLINE MAP
SHOWING
COMMERCIAL RELATIONS
OF
PACIFIC COAST PORTS
Scale of Miles
0 50 100 150 200
CANADA
MEXICO
PACIFIC OCEAN
Seattle
Tacoma
Portland
Spokane
Los Angeles
San Francisco
San Diego
Phoenix
Tucson
WHIDBY ISLAND
PUGET SOUND
Everett
Renton
Bremerton
SAN PABLO BAY
Vallejo
Oakland
Alameda
Berkeley
San Quentin
Pt. San Pedro
Pt. Richmond
Pt. Isabel
Pt. Bonita
Pt. Lobos
Pt. San Pedro
Pt. San Mateo
SUISUN BAY
SCALE FOR PORT MAPS
Scale of Miles
National City
North San Diego
Coronado
Point Loma
Soledad Mt.
La Jolla Pt.
Pt. Mugu
Santa Monica
Redondo
Wilmington
San Pedro Hill
Long Beach
Aberdeen
South Aberdeen
Hoquiam
GRAY'S HARBOR
South Bend
WILLAPA BAY
Astoria
Warrenton
VANCOUVER ISLAND
Victoria
Esquimalt
Gonzales Pt.
STRAIT OF JUAN DE FUCA
PENINSULA
Port Townsend
Discovery Pass
Admiralty Inlet
Vancouver B.C.
Westminster
Boundary Bay
BOUNDARY BAY
BRITISH COLUMBIA
UNITED STATES

present, the prophecy of one of her own poets, who has pictured the time:

"When through this Gate the treasures of the North
Flow outward to the sea."

Puget Sound.—Proceeding northward, the next great strategic point in the commerce of the Pacific Coast, and, in some respects, the most important of all, is Puget Sound. It is a vast inlet from the sea through a strait 15 miles wide and nearly 100 miles long, branching both north and south, in sheltered inland waters which abound in ideal natural harbors. So perfect are these harbors that their chief defect is an excess of what anywhere else would be considered almost the chief advantage, the depth in some being such that anchorage ground is scarce, and extending so close in shore as to make wharf building difficult and expensive. The tidal range is also large, with strong tidal currents, and the teredo is very active. Except for these drawbacks, the Sound harbors are ideal. Take Seattle, for instance. Into the spacious enclosure of Elliott Bay, a ship can enter without tug or pilot and pass directly to berth under her own steam in all conditions of tide, and in almost all conditions of weather, and always feel certain that her hull will not touch bottom. It is the most favored port in the world in this respect, excepting only one or two of its sister ports on the Sound, which enjoy the same advantage.

Nature has certainly lavished her richest favors on these ports and provided them with advantages which no expenditure of money can ever give to a port situated as is Portland or Los Angeles. But, as an offset to this great advantage, she has placed a handicap which is of very serious consequence; she has barred off Puget Sound from its great hinterland to the east by a massive mountain range, the lowest passes of which are more than 3 000 ft. high. These passes have been scaled successfully, it is true, and the terminal rail rates are the same as to Portland, but the handicap is there, nevertheless, and is the most serious commercial problem which Puget Sound faces to-day. It is as if the Catskill Range, with twice its actual height, extended along the coast in an unbroken wall from Boston to Baltimore, shutting out New York from its rightful hinterland. Local enthusiasts in both Portland and Seattle speak of their respective cities as the "New York of the Pacific." In truth, each is only half

New York; or rather, New York, in its natural advantages as a port, is the Seattle and Portland combined of the Atlantic. Its harbor rivals that of Seattle, while its Hudson River route to the interior rivals that of the Columbia. Puget Sound does not fully appreciate this difference. Seattle and Tacoma are spending millions in developing harbors which are already perfect beyond any in the world, but not a cent to overcome this barrier which stands between them and their hinterland. The writer once had the temerity to suggest that a low tunnel, serving all the railroads and giving practically a water-grade connection between the Sound and the Columbia Valley, was essential to the commercial supremacy of Puget Sound ports. Time alone will justify or discredit this prediction, but it is certain that with an outlay no greater than what has been expended on three of the Alpine tunnels, a line could be secured which would serve, on the most favored gradients and with shortest distances, the entire country from the Great Basin to the interior of Canada. Puget Sound is the natural entrepôt for the trade of Alaska, is a day nearer the ports of Japan and Northern China than San Francisco, and has the best harbors in the world. Only the natural barrier which separates it from its hinterland keeps it from playing the part which it should in the commerce of the Pacific.

We have considered Puget Sound as a great harbor whose entrance is the broad, deep Strait of Juan de Fuca. Inside the Strait there are numerous competing ports, of which Seattle is the central and most important. Thirty miles farther south is Tacoma, Seattle's most formidable competitor in United States waters. It has a harbor rivaling that of Seattle, and has been particularly favored by the great railway systems, which have given it an importance even beyond what its comparative advantages justify. Besides Tacoma there are the harbors of Everett, Anacortes, and Bellingham, all of which have splendid natural advantages and do a thriving trade. On the west side of the Sound is Port Townsend, once looked on as the coming port of Puget Sound, but now important mainly as the headquarters of the Customs District of the Sound and the Quarantine Station. It is on the wrong side ever to assume first importance.

All the great strategic points in United States territory, which we have considered, have been guarded by elaborate sea-coast defenses, while San Francisco and Puget Sound have been made great naval bases.

Vancouver, B. C.—Seattle's greatest rival for commercial supremacy north of Portland and Tacoma is Vancouver, B. C., a rival whose strength lies not so much in its natural advantages as in the artificial conditions arising from its being in a different national jurisdiction. It has a good strategic location, it is true, being at the outlet of the second greatest river of the Pacific Coast; but its harbor is inferior to those farther south, and its rail connections east, even to Central Canada, are inferior. Its early growth was due primarily to the fact that it was a Canadian port. That the port would have developed where, or as rapidly as, it has, if there had been no international boundary near by, may well be doubted. As conditions actually exist, however, taken with the present status of American navigation laws by which cheaper foreign shipping is excluded from our coastwise trade, Vancouver is a formidable competitor with ports farther south. In the fish and lumber export trade, particularly, conditions tell heavily in her favor.

Prince Rupert.—Prince Rupert is the northernmost Pacific terminus of the transcontinental lines, and will remain so until some line shall cross into the valley of the Yukon and descend that mighty river on its way to the westernmost apex of the Continent. It is 40 miles south of the international boundary of Alaska (54° 40′) and about 700 miles along the coast from Seattle and that much nearer Alaska. It is the westernmost, as well as the northernmost, transcontinental terminus, and is 500 miles (according to the statement of the Grand Trunk Pacific officials) nearer Asia than any other terminus on the Pacific. It is said that a traveler from China would be able to reach Winnipeg, *via* Prince Rupert, before he could reach Vancouver, if he were to go by that port. Add to this the fact that the gradient over the mountains, with the exception of about 20 miles of 1% on the west slope, is everywhere under five-tenths, and most of the way much less than this, and some of the physical advantages of the route are apparent. Add again the proximity of Prince Rupert to the limitless wheat fields of Canada, the fact that it is in the very center of the salmon and whale industry, among inestimable quantities of virgin timber, and its great future seems doubly assured.

Its advantages are offset to some extent by the severity of northern winters, but chiefly by the different nationality of the territory (Alaska) which it is best fitted to serve.

Secondary Ports.—Somewhat outside the rivalries of the great ports along the Coast, and flourishing on advantages which are peculiar to themselves, are several smaller ports. Among them, and the more prominent, are San Luis Obispo, about half way between San Francisco and Los Angeles; Humboldt Bay, on the North California Coast; Coos Bay, 200 miles south of the Columbia and serving an important section of Western Oregon; Astoria, just inside the Columbia Bar and the first port to be established on the North Pacific Coast; Grays Harbor, a great lumber port on the west coast of Washington, 45 miles north of the Columbia; and Victoria, an important harbor and naval base on Vancouver Island. There are numerous harbors on the Alaskan coast, but they are still in a state of Nature, as very little has been done toward converting them into up-to-date ports. The prospective opening of the coal fields of Alaska, and a more definite Alaskan policy on the part of the Government, will undoubtedly lead to the establishment of permanent facilities at some of these points in the near future.

II.—Descriptive Data.

In its physical characteristics the Pacific Coast line of North America increases in severity from south to north, but there is less diversity in climatic conditions than one might expect, for such great differences in latitude, owing to the moderating effect of the ocean currents. It is not until well up on the Alaskan Coast that one finds the harbors regularly sealed by ice in the winter season. Storms are more severe in the northern latitudes, but this is more than offset, as far as coastwise trade is concerned, by the sheltered inland passages which extend for at least 1 000 miles from Southern Puget Sound north. The tidal fluctuation increases from south to north from a mean of about 4 ft. at San Diego to 14 ft. at Prince Rupert. The teredo is very destructive in all the waters of the Coast, but the limnoria is active only in Californian waters. Nearly all the tributary streams are heavy silt carriers, and the primeval bays and inlets are partly or wholly filled up, making dredging a necessary adjunct of harbor development all along the Coast, and necessitating costly training dikes to scour channels across bars or shoals. In Southern California, the immediate shores were originally lightly timbered; but from Northern California north they were covered with magnificent forests. The coast line is re-

markably uniform, and unbroken by indentations as far as the Strait of Juan de Fuca, but from there north the exact opposite is the case.

With these few observations on the broad physical characteristics of the Coast, more detailed consideration will be given to the several ports.

San Francisco Bay.—The Bay of San Francisco (including, in that term, San Pablo and Suisun Bays) has a total area of 420 sq. miles and a shore line of about 350 miles. The area exceeding 30 ft. in depth at low water is about 190 sq. miles. The extreme tidal range is 8 ft., and the mean is 4.3 ft. The tidal currents are strong in the Golden Gate and Carquinez Straits, amounting to 7 miles per hour at spring tides. The entrance to the Bay is about 1 mile wide and very deep, and is guarded by formidable defenses. In a half circle about the entrance on the ocean side, and some 6 miles distant, is a narrow bar with a ruling depth of 30 ft. at low tide and two crossings of over 40 ft.

The heavily silt-laden streams—the Sacramento and San Joaquin, with a water-shed of more than 60 000 sq. miles, extending 400 miles along the western slope of the Sierra—have brought down immense deposits into the Bay and are progressively reducing its area. It once extended far up the valley of each stream, but is now practically limited to the area below the Straits of Carquinez, for Suisun Bay is shoaled up so as to be of little use for navigation except on the through channel to the rivers above. The deposits have also invaded the Lower Bay and have shoaled the depth, over two-thirds of its area, to less than 18 ft. Due probably to the action of the inflowing tides, this shoaling has been forced mainly on to the east side of San Francisco Bay and the north side of San Pablo Bay, leaving the east shore, where Oakland, Berkeley, and Alameda now stand, quite unapproachable in their natural condition by any craft larger than a rowboat, while the shore of the San Francisco Peninsula has practicable depths for the largest shipping close in to the bank. It was mainly this condition that made the Port of San Francisco Bay develop where it did instead of on the east shore. Now that reclamation work is utilizing the shoal areas to make new lands by dredging out deep slips, shipping will gravitate more and more to the east shore in order to avoid ferry inconvenience.

The shoaling of San Francisco Bay is one of those great natural blessings which the unthinking are so accustomed to look on as a curse. One-tenth of its natural area, with deep connecting channels,

would serve every possible need of commerce, while the other nine-tenths would be of immeasurably greater benefit reclaimed and turned to industrial or agricultural use. Every cubic yard of earth washed down from the rugged slopes of the mountains is worth a thousand times more in those low areas, where it is turned to efficient use in the service of Man.

San Francisco Bay is perfectly sheltered from ocean storms, is not subject to flood effects except in the extreme upper portions, at the outlet of the Sacramento and San Joaquin Rivers, and is absolutely free from ice. There were originally several dangerous rocks in the channels, but these, for the most part, have been removed, the most important removal being that of Blossom Rock. In practically all respects the natural advantages of San Francisco Bay are of the highest order, and, if its human custodians are faithful to their trust, it is destined to remain for a long time the leading port on the Pacific Coast.

San Francisco proper and its port grew up on the narrow peninsula lying between the southern half of the Bay and the ocean, south of Golden Gate. As far as land communication to the north and east is concerned, the city is practically an island, and has to rely exclusively on ferry service. The water-front development extends from the northeast extremity of the peninsula around into the Bay, and as far south as the city and county boundary. The jurisdiction of the Board of Harbor Commissioners stops short at this point, which fact is clearly indicated on Plate X. At present the berthing space for all classes of vessels aggregates more than 10.5 miles. The several ferries to the east shore and other points land near the foot of Market Street, and carry 125 000 passengers daily. Exclusive of fairways or forbidden anchorage, there is, approximately, 100 sq. miles of available anchorage ground in the Bay.

Dock construction consists of three classes: Entire wooden construction, concrete subconstruction only, and complete concrete construction. Much trouble has been experienced with subaqueous concrete work, probably due to defective methods or inspection. Nearly all the sheds are built of wood, and their areas vary from 60 000 to 84 500 sq. ft. The State owns no dry docks, but there are two privately owned graving docks at Hunter's Point and two floating docks at the Union Iron Works Plant, besides the Naval Dock at Mare Island.

San Francisco is the only port on the Coast at which any great

amount of permanent sea-wall construction has been done. Work of this character has been carried on for the past twenty years. In that time, including work now in progress, nearly 13 000 lin. ft. have been built, at costs ranging from a little less than $100 to about $270 per lin. ft. The construction of the sea wall has resulted in reclaiming more than 25 acres, from which the annual rental now amounts to about $1 000 000, or considerably more than half the total revenue of the port.

The port's funds are derived from its regular revenues (rents, wharfage, dockage, tolls, etc.) and the sale of bonds. The regular revenues, from the beginning of the Board's operations in 1863, amount in round numbers to $28 500 000, and the funds from other sources, mainly bond sales, including recent issues still unexpended, to about $12 000 000. The expenditures for construction and repair amount approximately to $22 000 000, exclusive of the recent $10 000 000 bond issue. The running expenses of the port (salaries, law fees, etc.) amount to an average of 21.4% of all the disbursements. The total Federal appropriations for removing obstructions from the harbor amount to $516 000, so that the total outlay on the harbor, including recent bond issues, will be about $32 500 000. This, of course, is not all represented in actual present results, for much of it was for early work which has since been replaced, existing work probably not representing a present cost exceeding $10 000 000.

It is important to note that approximately one-half (of late years more than one-half) the revenue comes from ground rents, without which much larger bond issues would have been required.

The commerce of the port is very extensive. During the fiscal year 1908-09, 39 251 vessels of all descriptions were docked at the State wharves, and the short tons of freight passing over the wharves in the same period was 6 325 000. The Annual Report of the Engineer Department of the United States Army for the calendar year 1910 gives a total short-tonnage of 7 325 000, valued at $222 500 000, distributed as shown in Table 1.

The trade of San Francisco embraces every class of commerce that passes over the Pacific Ocean. The Government Transport Service on that ocean largely centers there, and the chief naval base of the Pacific Coast is at Mare Island in San Pablo Bay. In many other respects San Francisco stands far in the lead of any other port on the Coast.

TABLE 1.—FREIGHT TRAFFIC.

Articles.	Amount, in short tons.	Valuation.
Flour	98 089	$6 670 052
Corn	6 320	221 200
Wheat	143 104	5 724 160
Barley	543 407	16 302 210
Oats	29 869	896 070
Beans	46 913	3 753 040
Hay	214 734	4 079 946
Onions	16 815	1 325 200
Bran	16 865	573 410
Potatoes	98 415	3 936 600
Lumber	1 599 275	20 790 575
Shingles	34 037	1 157 258
Brick, cement, lime, etc	62 814	1 307 088
Coal, coke, etc	623 174	7 276 328
Crude oil	2 786 980	13 934 900
Paper	22 107	2 210 700
Wines, liquors, etc	101 269	20 253 800
Fertilizer	25 170	880 610
Salmon, codfish, etc	89 180	15 723 656
Butter, eggs, cheese, etc	26 984	11 436 440
Fruits, dried fruits, etc	86 553	10 470 950
Sugar, tea, coffee, canned goods	295 089	38 342 350
Silk, cotton, dry goods, leather	31 159	13 370 800
Wool, gunnies, matting, jute, etc	29 182	5 836 400
Rice, salt, coal oil, etc	33 709	1 849 205
Railroad ties, pig iron, etc	116 302	1 528 762
Miscellaneous	147 062	12 676 488
Total	7 324 577	$222 478 148

Oakland and Alameda.—The physical characteristics of San Francisco Bay, of which Oakland and Alameda occupy the east shore, have already been described. The distinctive east shore harbor is the San Antonio Estuary, which lies between Oakland and Alameda and extends entirely around the land side of the island on which Alameda is situated. It is referred to in Government reports as Oakland's inner harbor. A channel leads in, from deep water in the Bay, between jetties 800 ft. apart, which also serve as moles with ferry slips at the ends. The channel, as developed under the Government project now in force, will be 500 ft. wide and 30 ft. deep to a large "tidal basin" about opposite the center of Alameda, with a channel 300 ft. wide and 25 ft. deep entirely around the basin, and a depth of 18 ft. in the "tidal canal" connecting the basin with San Leandro Bay, an inlet of San Francisco Bay on the southeast end of Alameda Island. This magnificent inner harbor has a shore line on each side of about 7 miles. On the Oakland side there is approximately 10 000 ft. of berthing space for deep-sea vessels and 16 000 ft. for smaller craft. The harbor is a great industrial center, some of the largest plants on the Bay being located there. There are four dry docks.

From the point of junction of the Estuary with the Bay, the shore line to the north is occupied by transcontinental railroad terminals and shops of ferry lines. In this stretch of about 2½ miles frontage is the so-called western water-front on which the Municipal Government is making improvements.

The port's revenues come from tax levies, bond sales, rentals, fees, etc. The City Government has recently issued $2 500 000 in bonds, and contemplates duplicating this issue in the near future.

The commerce of the port is large and varied, amounting, in 1910, to 3 575 000 short tons, valued at about $140 000 000, of which approximately one-half was carried in merchant vessels and one-half in ferries.

All along the east shore, and at many other points on both shores of the Bay and of San Pablo and Suisun Bays, there are wharves and industrial establishments of one kind or another, of which only mention can be made here.

Los Angeles.—As already observed, Los Angeles was less favored by Nature as a port than any other important locality on the Coast, and the making of a great port there is the work of its enlightened citizenship. The harbor was an open bay protected only from westerly and northerly winds. It lay wide open to an exposure of 120° toward the south except for the meager influence of Catalina Island, 18 miles in the offing. From this bay, through a long crescent-shaped bar, formerly called Rattlesnake Island, now Terminal Island, there opened inland, by a channel only 3 ft. deep at low tide, what has come to be under development the inner harbor of Los Angeles. It is really the delta of Los Angeles and San Gabriel Rivers (erratic streams characteristic of the water-courses of that section of the country) which bring down heavy loads of silt in their periods of sudden floods. The outer harbor, being exposed to storms from the southwest, required artificial protection, and this has been provided by one of the largest breakwaters in the United States, extending from Point Fermin, previously referred to, easterly, a distance of 11 000 ft., to a low-tide depth of 48 ft. The space thus enclosed, measured back from a line drawn from the end of the breakwater perpendicularly to the shore, is 960 acres, and of this more than 400 acres has a depth of over 30 ft. The anchorage is good, the kelp which formerly was a great annoyance has died out, and the harbor thus formed serves its purpose excellently well. The mean tidal range is 4.1 ft., and the extreme 10 ft.

Great reliance for port facilities, however, is being placed on the inner harbor, which is almost wholly an artificial creation. Its entrance from the outer harbor is completely sheltered by the breakwater. Beginning with 700 ft., it widens to 1 000 ft. at a point 3 000 ft. in, and then rapidly narrows to 500 ft., which width it retains to a point 7 500 ft. in from the outer end. The width then increases somewhat, and the channel terminates at a distance of 12 000 ft. from the entrance in a turning basin 1 600 ft. in diameter. From the turning basin channels lead off to the main interior docks known as the East and West Basins, respectively, which, with their future ramifications, will become the center of Los Angeles Harbor. The situation can be better grasped by reference to Plate X. The present width and depth are 400 ft. and 20 ft. to "the foot of the wharves"; thence 25.5 ft. to the turning basin. The existing Government project contemplates a depth of 30 ft. up to and including the turning basin, and two channels, 200 ft. wide and 20 ft. deep, from the turning basin to the East and West Basins.

As already stated, the harbor is almost wholly of artificial creation, and it is costing vast sums, entirely apart from docks and wharves, to arrive at a point which Nature unaided has given to most of the other ports on the Coast. Los Angeles, though liberally aided by the Federal Government, is helping herself effectively. It is a fine example of a virile civic spirit refusing to be balked by Nature in the matter of access to the sea. It will eventually rank with such ports as Hamburg and Glasgow in the extent to which Man by his energy and persistence has overcome the deficiencies of Nature.

The total development of berthing space is about 19 000 ft., most of which is directly controlled by the railroads. There is no dry dock. The wharves are principally of timber construction, but in recent years there has been a tendency to extend the use of concrete and steel, and the recent controversy between the Board of Public Works and the Board of Harbor Commissioners turned largely on the choice between concrete and creosote piling for the substructures. There is no seawall construction, unless the great breakwater be classed as such.

The local funds for port development, so far, have come entirely from bond issues, but ground rent will doubtless become a source of revenue in the near future, together with the revenues from docks which the port authorities are planning to build. Local expenditure to

date, including that of the Village of Wilmington prior to annexation to Los Angeles, and including also outstanding contracts, amounts to $625 000. The City has voted $3 000 000 in bonds, and it gave a pledge to the Cities of Wilmington and San Pedro prior to annexation that it would expend not less than $10 000 000 in port development. The expenditure by the United States on the outer and inner harbors, including outside authorizations, is:

Outer harbor	$3 078 000
Inner harbor	2 308 000
Total	$5 386 000

A great saving fact in the enormous labor of building the Harbor of Los Angeles is its close relation to the industrial development of the city. The lands into which the waterways are being dredged are admirably adapted for factory locations, and the material of excavation is being used in making the necessary fills. The whole development goes hand in hand in a way to produce the best results.

The trade of Los Angeles is active and is steadily increasing. The principal articles of commerce are oil, cement, and lumber products. The estimated short-tonnage and valuation for the calendar year 1910 are given in Table 2, the totals being 1 709 000 tons valued at $47 000 000.

As yet no public terminal or belt-railway system has been constructed in Los Angeles, but the city is taking steps to acquire one which will serve the port and connect it with the city.

San Diego.—Nature has provided for San Diego, in the first instance, what Los Angeles with the expenditure of many millions can scarcely expect ever to equal. In general exterior outline the two ports are quite similar. Coronado Beach corresponds to Terminal (old Rattlesnake) Island, Point Loma to Point Fermin, while the entrances to the inner harbors are similarly situated in both cases. The difference is that at San Diego the entrance had a very good depth and ample shelter to begin with, while the inner harbor was already dredged. At Los Angeles there was a depth of 3 ft. at the entrance and no harbor to speak of. If San Diego had been as favorably located with respect to its hinterland as Los Angeles, the natural advantages of her harbor would have been decisive, always provided a virile and wide-awake people were charged with its development.

TABLE 2.—Freight Traffic.

Articles.	Amount (customary units).	Amount, in short tons.	Estimated valuation.
Lumber	579 842 876 ft. b.m.	1 014 724	$12 756 543
Shingles	243 595 000 pieces	48 719	487 190
Shakes	4 204 000 pieces	2 803	60 958
Lath	70 460 000 pieces	19 572	147 966
Ties	696 308 pieces	44 563	445 640
Piles	18 317 pieces	17 584	366 340
Poles	43 526 pieces	36 561	652 890
Posts	79 565 pieces	2 387	59 673
Logs	7 940 pieces	5 558	55 580
Doors	10 578	529	21 156
Staves		525	15 750
Lumber trucks	182	75	4 550
Grain		30 700	921 000
Flour		3 523	176 150
Sugar		2 028	202 800
Paper		4 600	391 000
Asphalt		9 587	67 109
Coal and coke		6 976	71 390
Cement	225 350 bbl.	45 070	450 070
Pig iron		3 454	51 810
Oils	1 301 302 bbl.	259 467	2 138 571
Wool	311 bales	77	23 100
Plaster	79 600 bags	3 748	37 480
Salt	741 bags	78	2 028
Rice	6 170 bags	308	20 533
Brick	377 000 pieces	942	3 328
Chemicals, drugs, etc		488	97 600
Manufactured iron and steel		5 964	1 192 800
Fruits and nuts		1 399	279 800
Canned goods	9 560 cases	239	26 680
Fertilizers		5 010	143 886
Hay and feed		159	2 385
Scrap metals		1 702	34 040
Wines and liquors		424	84 800
Live stock	11 211 head	3 080	213 792
General merchandise		126 671	25 334 200
Total		1 709 294	$47 040 588

The area of San Diego Bay inside the entrance is 24 sq. miles, of which 1 600 acres is more than 30 ft. deep; and the length of shore line is 38 miles. The original depth of 21 ft. in the entrance has been increased by the Government to 30 ft., and the present project contemplates an enlarged channel of this depth 600 ft. wide. The present berthing space amounts to 4 500 lin. ft., with a low-water depth of 22 ft. All the piers are private property, but they are subject to sequestration by the City at any time on payment of actual value. There is no dry dock and no public terminal railway. There is a marine railway for vessels up to 1 000 tons burden. The Government maintains a torpedo and submarine boat station in the harbor, together with quarantine and coaling stations. There is an important ferry service between the city and Coronado Beach.

The present sources of revenue to the port are bond sales, of which only one (for $1 000 000, voted November, 1911) has ever been au-

thorized. The total Government appropriation for the improvement of the harbor is $810 000.

The commercial business of the port is shown in Table 3, which embraces the calendar year, 1910. The totals are 400 000 tons, valued at $22 500 000.

TABLE 3.—Freight Traffic.

Articles.	Amount (customary units).	Amount, in short tons.	Estimated valuation.
Lumber	80 066 000 ft. b. m.	140 115	$1 761 452
Crude oil	466 888 bbl.	88 419	350 166
Cement	15 234 bbl.	3 047	30 468
Wool	34 000 lb.	17	6 800
Salt	.....	392	10 192
Ores	.....	2 714	542 800
Manufactured iron and steel	.....	1 811	362 200
Brick	618 160 pieces	1 545	4 950
Grain	.....	445	13 350
Fertilizer	.....	1 634	42 484
Coal and coke	.....	38 954	154 057
Live stock	11 043 head	3 036	147 984
Pig iron	.....	2 961	44 085
Onyx	.....	16 510	119 430
Sulphate of potash	.....	1 774	66 269
General merchandise	.....	94 674	18 934 800
Total	.....	398 048	$22 591 487

Secondary Harbors South of the Columbia River.—Between Los Angeles and San Francisco is the important secondary harbor of San Luis Obispo; and between San Francisco and the mouth of the Columbia are the important secondary harbors of Humboldt Bay, California, and Coos Bay, Ore. These harbors, roughly speaking, divide by equal intervals the long stretch of coast line between the principal ports. At San Luis Obispo the development work by the Government is in the nature of a breakwater to afford shelter; in the two northern harbors, it consists of jetty construction designed to deepen the natural channels over the bars. In all three ports the outlay has been heavy in proportion to the commerce. Some of the most successful jetty work on the Coast is at Humboldt and Coos Bays, but it is impossible, in this paper, to make more than a passing reference to it.

The Port of Portland.—This is the only port of those considered in this paper which, in its physical characteristics, is strictly a river port. It is so far inland that tidal fluctuation, though perceptible at periods of low water, is negligible. On the other hand, it is subject to all the fluctuations of level characteristic of rivers above tidal influence, and the range between extreme high and extreme low water

is more than 30 ft. The harbor proper is not on the Columbia, but on the Willamette, about ten miles above its mouth, though, in general terms, looking to the future, the whole of both streams, from Vancouver, Wash., to Portland, may be regarded as the port of the Columbia Valley. Being a river harbor, the whole trend of its development shapes itself to that fact, and its counterpart is to be found in such a port as New Orleans rather than in those which have been considered in this paper. Quay development, parallel to the current, is the rule, instead of slips or piers projecting into the stream. While diurnal fluctuations are absent, and no provision therefor is necessary in dock construction, the great seasonal fluctuations introduce even more serious difficulties in providing for differences of level.

The length of available berthing space for deep-sea shipping is about 6 600 ft. on the east side and 8 500 ft. on the west side. The corresponding figures for river boats and the mosquito fleet are 4 725 and 6 040 ft., respectively. Railroads own 25 800 ft. of water-front on the west side and 17 150 ft. on the east side. The port has two floating dry docks, but there is no municipal or belt railway.

As is often the case with a port situated far inland on a river, the problem of providing an adequate channel to the sea is the most troublesome. With 110 miles of river channel in which a low-water depth of 10 ft. was all that could be depended on originally, and with a wild and tempestuous bar at the river mouth over which the natural depth was only 20 ft., the problem of securing and maintaining a channel was the all-important one. The work on the bar must take rank among the greatest, if not the greatest, of the kind ever constructed. It has all been done by the Government. The channel improvement from the bar to Portland has been done jointly by the Government and the port in proportions of expenditure as shown on the following page. There is at present a depth of 26 ft. at low water, and the Government is about to commence work to increase this to 30 ft. The Government now has three dredges available for the work, and the Port of Portland has two, and it is expected that this fleet will be soon increased by two more.

The port's revenues come from tax levies, bond sales, dry-dock charges, dredging, and a few minor sources. Nothing is derived from dockage, wharfage, and similar charges. The total amount of funds provided by the Government and locally is approximately as follows:

By the Government—The Columbia Bar	$10 145 000	
" " " River Channel to Portland	2 725 000	$12 870 000
By the Port of Portland, approximately		3 700 000
By the Portland Dock Commission (bonds voted, but not yet expended)		2 500 000
Total		$19 070 000

As already stated, Portland is admirably situated on the land side to develop a large port business, but the channel difficulties between the port and the sea somewhat offset this advantage. The commerce, nevertheless, is large and varied, as will be seen from Table 4, the totals being:

Exports and imports	415 522	short tons,	valued at	$8 451 414
Domestic	2 025 434	" "	" "	31 313 990
Local	5 393 317	" "	" "	25 116 602
Totals	7 834 273	short tons,	valued at	$64 882 006

Grays Harbor.—The most important of the secondary harbors of the Coast is Grays Harbor, forty-five miles north of the Columbia. It is a pear-shaped body of water nearly 100 sq. miles in area. It is really a vast mud flat, much of it being bare at low tide, and the channel to the upper portion where the Towns of Hoquiam, Aberdeen, and Montesano are located, has to be maintained by dredging. Extensive jetty work has been built and is now in progress to develop a channel over the bar. The port has three railroads. Its importance depends primarily on its lumber trade, and it is the largest cargo lumber port in the world. The Government has appropriated about $2 400 000 for improvement of the harbor and bar. The commerce for 1910 amounted to 670 000 tons, valued at $6 000 000 of which more than 80% represented lumber products.

Puget Sound.—In their physical characteristics, the Puget Sound ports are much alike, the water being very deep, anchorage ground scarce owing to depth, the tidal range large, and the teredo active. As a rule, there is absolute freedom from obstructions in the channels of approach. Pilots and tugboats (except the latter for sail boats)

are unnecessary, though pilotage on the larger vessels is customary. In all cases the streams leading into the harbors from the Cascades bear heavy loads of silt, and have filled up the estuaries for long distances. The tide flats thus built up have proven an invaluable asset, for they have made up to some extent the deficiency of low flat lands necessary for industries and terminal facilities. South of Deception Pass rock is scarce, but north of that point it outcrops everywhere, and in some instances forms dangerous obstructions. The whole

TABLE 4.—FREIGHT TRAFFIC.

Articles.	Amount (customary units).	Amount, in short tons.	Valuation.	Rate per ton-mile.
EXPORTS AND IMPORTS.				Mills.
Bags and burlap	5 230 tons	5 230	$488 934	0.694
Cement	80 852 bbl.	19 202	119 061	0.122
Coal	18 733 tons	18 733	47 313	0.694
Coke	4 021 tons	4 021	14 187	0.308
Fibers	1 703 tons	1 703	190 313	0.793
Flour	283 679 bbl.	27 800	1 186 654	0.638
Fire brick and clay	7 149 tons	7 149	33 848	0.191
Iron (pig and bar)	8 774 tons	8 774	137 454	0.184
Lumber	122 711 000 ft. b. m.	159 524	1 505 497	0.620
Machinery	50 tons	50	1 445	0.694
Rice	2 082 tons	2 082	90 662	0.851
Sulphur	7 630 tons	7 630	124 643	0.638
Wheat	5 120 826 bushels.	153 624	4 511 403	0.421
Total		415 522	$8 451 414	
DOMESTIC.				
Lumber	317 932 977 ft. b. m.	413 313	$4 133 130	3.331
Logs	48 000 000 ft. b. m.	96 000	288 000	1.50
Oil	6 900 000 bbl.	1 159 200	5 477 600	2.50
Miscellaneous	356 921 tons	356 921	21 415 260	5.00
Total		2 025 434	$31 313 990	
LOCAL.				
Coal	3 424 tons	3 424	$23 968	20.00
Fish	13 223 tons	13 223	1 586 760	20.00
Flour	107 397 bbl.	10 525	526 250	20.00
Fruit	3 478 tons	3 478	208 680	40.00
Grain	728 413 bushels.	18 116	391 525	25.00
Hay and feed	11 483 tons	11 483	172 245	20.00
Hops	135 tons	135	27 000	50.00
Live stock	22 001 heads.	4 592	918 400	35.00
Logs (towed)	1 035 295 704 ft. b. m.	2 070 591	6 211 773	5.00
Lumber	13 930 960 ft. b. m.	18 110	181 100	20.00
Manufactured iron and steel	7 369 tons	7 369	257 915	25.00
Merchandise	55 146 tons	55 146	5 514 600	50.00
Miscellaneous	70 247 tons	70 247	4 214 820	50.00
Oil (barged)	1 872 582 bbl.	314 594	1 343 782	30.00
Piles (towed)	14 017 887 lin. ft.	280 358	1 121 432	12.50
Potatoes	30 610 000 lb	15 305	306 100	30.00
Sand and gravel (barged)	1 059 133 cu. yd.	1 592 671	796 335	8.00
Shingles	2 193 733 bundles.	38 390	1 151 700	28.00
Stone (barged)	562 364 cu. yd.	843 550	84 355	10.00
Wood (barged)	14 603 cords	21 905	54 762	15.00
Wool	105 tons	105	23 100	50.00
Total		5 393 317	$25 116 602	

conformation of the Puget Sound country is the result of glacial action, and many of the lakes and bays are simply the gouged out paths of glaciers, while the shores are masses of the most heterogenous material. So irregular and broken with indentations and islands is the Sound area that tidal currents and stages are wholly irregular. Tidal predictions are possible only as a result of long observation, and are stated in the Government tables in terms of retardation behind the Port Townsend prediction.

With this general reference to the ports of Puget Sound, a somewhat detailed description will be given of its principal port, Seattle, Wash. Seattle Harbor means primarily Elliott Bay, a not very large indentation into the east shore of the Sound. Between its extreme limits, West Point and Alkali Point, the pier-head line is 10 miles long. The really practical limits of the harbor, as far as the near future is concerned, may better be taken as Duwamish Head and the southern point of Magnolia Bluff, directly opposite. Within the line joining these points is Elliott Bay proper, quite similar in contour (though slightly smaller in area) to Commencement Bay, on which the port of Tacoma is situated. The area of Elliott Bay, as thus limited, is 4.3 sq. miles, and its shore line (pier-head line), not counting slips and waterways, is 6.5 miles. Over this entire area there is ample depth for the heaviest shipping. There is never any ice or rough weather to disturb deep-sea vessels, though southwest winds disturb small vessels in the reach south of Smith's Cove, as north winds do those on the south side of the harbor. The tidal currents are too slight to be drawbacks of any moment.

The water-front development is most pronounced along that portion where the city began its growth. For upward of a mile the entire frontage is in active use. There are some important piers, which, as a rule, are run out at very oblique angles with the shore in order to secure the necessary length without getting out to prohibitory depths. At the north is the important pier of the Great Northern Railway, and near it a still unimproved waterway, Smith's Cove, 1 mile long and 400 ft. wide, which is destined to important use in the growth of that part of the city. On the south side of the harbor, the delta of the Duwamish River has been filled in, except for the reservation of two waterways, the East and the West, inclosing a tract of land popularly known as Harbor Island, from which great things are expected in

the future growth of the city. As the plan will finally work out, East Waterway will be practically a slip, about 1 mile long and 750 ft. wide, with slips and quays on the two sides. The east side of the Waterway is really a part of the main water-front. West Waterway will be 900 ft. wide and will be the outlet of the Duwamish River.

The valley of the Duwamish furnishes the single large tract of bottom land directly adjacent to the harbor, and it must naturally develop into an important industrial center. Plans are perfected and well under way, with a large part of the funds already provided, for straightening the river and excavating a channel 4 miles up the valley to a bottom width of 150 ft. and a low-water depth of 16 ft. The utility of this work in the industrial development of the city is bound to be great.

Such is what may be termed Seattle's outer, or tidal, harbor, with a perimeter, including the three main waterways, of 12.5 miles east of Magnolia Bluff and Duwamish Head, and this is susceptible of almost indefinite extension by pier and slip construction. Every foot of this frontage is served, or readily capable of being served, by railroads, and the possibilities of expansion are all that are likely to be required for a century to come. The extent of existing wharf construction is such as to give about 14 500 lin. ft. of berthing space.

In addition to this magnificent gift of Nature, Seattle has another absolutely unique among the sea ports of the world—what may be called her inner harbor, soon to be opened to the commerce of the seas. The glaciers, in carving out the picturesque topography of this city, left two remarkable troughs or basins, bordered by hills from 200 to 500 ft. high. These basins are just enough above sea level to keep out the tides, and their waters, therefore, are fresh. Their own levels differ by only about 10 ft., the larger lake being the higher. Nearly all the area of both lakes is deep enough for the heaviest shipping; and all that Nature has left undone to make it accessible from the sea is to provide a lock and an artificial channel from Lake Union to the Sound, and a channel between the lakes. This work is being done by the Government and the local community. A channel, 100 ft. wide on the bottom and 36 ft. deep, will be provided, with two parallel locks, one 825 ft. long and 80 ft. wide, interior dimensions, and the other 150 ft. long and 30 ft. wide, for small boats. Salmon Bay, a tidal slough between Lake Union and the Sound, will be raised by the lock

and dam to the level of Lake Union, and Lake Washington will be lowered to the same level, thus giving a uniform level from the lock to the remotest corners of Lake Washington. The canal right of way will admit of double the present proposed width, should commerce ever demand it. The bridges will have 30 ft. clearance, except one railroad bridge which will remain open when not in use. The fluctuation of level will not exceed 2 ft. The lakes are free from obstructions and do not freeze. Incidentally, the lowering of Lake Washington will absorb the floods of its water-shed and of Cedar River, and thus furnish a needed relief to the floods of the Duwamish, besides draining extensive marsh areas around the shore and making them available for industrial use.

There has been much speculation as to the value of this proposed improvement work. On the face of it, it would seem to furnish one of the most ideal opportunities that any port ever had. Think what it would mean to Los Angeles, if, instead of spending tens of millions for a narrow and restricted tidal harbor, she could, at an expense of $4 000 000 or $5 000 000, open up 40 sq. miles of fresh, deep water with a shore line of 100 miles. But at Seattle it does not mean what it would at Los Angeles. The rare perfection of the harbor of Elliott Bay places even this wonderful fresh-water harbor at a discount, as far as over-seas shipping is concerned. Still, its incomparable advantages are bound to tell. Absolute freedom from tidal fluctuation and currents, freedom from the teredo and, therefore, assurance of permanence in subaqueous timber construction, the beneficial effect of fresh water on the hulls of vessels, and the access which the lakes give to all sections of this district, are advantages which will not go unused. The first and immediate effect will be on small-boat traffic, lighterage, and the like, and a complete readjustment of this class of traffic may surely be expected. Salmon Bay and Lake Union will become great docks in the very heart of the city, while Lake Washington will offer thousands of acres of shore lands adapted to manufacturing. Gradually, larger shipping will follow, particularly in quest of lumber cargoes. Already inquiries are being made for sites for the fishermen's fleet and the yachting and motor-boat fleets. The large number of vessels which ply these northern waters, but which lie idle in winter, will find ideal refuge here. It will be strange if even the Navy Department does not take advantage of these fresh,

sheltered waters to lay up such of its vessels on this Coast as remain for a time out of commission. The ship-building industry of the entire Northwest ought to center at Seattle. Therefore, in spite of the unrivaled attractions of Elliott Bay, Seattle's future fresh-water harbor seems bound to become a priceless adjunct to the city's growth.

The natural perfection of Seattle's outer harbor is in no way better shown than by the remarkable fact—probably unique among American harbors—that no Federal appropriation has ever been made for its improvement. The only expenditure, not by direct private initiative, has been the development of the East and West Waterways and Harbor Island, and this has cost, in round numbers, $3 000 000.

The development of the inner harbor will have cost, when completed, exclusive of bridges and other crossings, approximately $4 250 000, of which the Government will have contributed about $2 750 000, and local agencies (State and County) about $1 500 000.

On the west shore of the Sound, directly opposite Seattle, and concealed in a deep inlet, with narrow, tortuous approaches, yet safe for the heaviest ships and protected by strong defenses, is one of the most interesting spots in the city. It is the Naval Station of the North Pacific Coast, destined to expansion, because of its superior advantages, into one of the greatest, if not the greatest, in the United States. It is mentioned in this connection with particular reference to its dry-dock facilities, which are often utilized by commercial vessels. There are other dry docks on the Sound, particularly the Heffernan Dock on East Waterway, Seattle, but no others of a capacity for docking the largest vessels.

Tacoma, the second port in importance on the Sound, is 30 miles south of Seattle, on Commencement Bay, similar in form and position to Elliott Bay. The tide flats have been built up by deposits from the Puyallup River, as those of Elliott Bay were by the Duwamish. Two waterways have been laid out, and one of them, the City Waterway, has been excavated into these flats. An attempt was made to excavate the other—the Puyallup Waterway—but the excessive deposits of silt brought down by the river (greatly augmented since 1906 by the bodily diversion to the Puyallup, through natural causes, of the White River from its former outlet into the Duwamish) made it impossible to secure or maintain the desired depth. The principal dockage facilities consist of a long quay dock on the west

side of the Bay, the City Waterway, and a slip excavated by the Chicago, Milwaukee and Puget Sound Railroad for its own use. The aggregate berthing space is 14 500 lin. ft. The business of the port has been greatly promoted by the location there of the principal shipping facilities of two important railway systems, and it now crowds closely on that of Seattle.

There are five other ports on the Sound worthy of particular note. These are Olympia, at the extreme southern end, and formerly important as the first point where travel from the Columbia touched the Sound—a very important circumstance before there were any good roads or railroads in the country. Everett is 30 miles north of Seattle, and is at the point where the Great Northern Railway reaches tidewater. It lies at the mouth of the Snohomish River, and is a port of considerable importance. Bellingham is another of the important secondary ports, and the farthest north of those considered in the paper. Anacortes is a flourishing port on Fidalgo Island, reached by a branch line of the Great Northern. Port Townsend is the chief port on the Olympic Peninsula, and has a peculiar importance quite apart from its commercial business, which is slight. It stands at the gateway to Puget Sound, where the great defenses are, it is the headquarters of the Customs District of the Sound, the location of the Quarantine Station and Marine Hospital, the base for nearly all marine reckonings pertaining to the Sound, and, in other respects, an artificially important port. Its location is such, however, that, despite its excellent harbor, there is no reason to expect that it will ever develop an important commerce of its own.

The expenditures, in round numbers, already made or authorized, by the Government in improving the several harbors of the Sound, are as follows. There have also been local expenditures in most of them, but these amounts, except in the case of Seattle, are difficult to determine.

Olympia	$160 000
Tacoma	320 000
Seattle (Canal)	2 750 000
Everett	700 000
Bellingham	60 000
Total	$3 990 000

The commercial statistics of the Sound ports are given in Table 5, the totals being as follows:

Port.	Short tons.	Valuation.
Seattle	2 371 628	$96 048 996
Tacoma	1 299 206	61 315 442
Other ports	2 244 889	28 892 495
Entire Sound	5 915 723	$186 256 933

TABLE 5.—FREIGHT TRAFFIC, 1910.

Article.	Amount (customary units).	Amount, short tons.	Valuation.
PORT OF SEATTLE.*			
Lumber	100 795 173 ft. b. m.	201 590	$1 562 567
Grain	1 675 745 bushels.	50 272	1 501 595
Flour	616 310 bbl.	60 398	2 619 887
Fodder		30 109	722 925
Fruit	148 219 cases.	3 705	256 072
Coal		637 851	2 706 147
Building material		†110 292	1 475 480
Merchandise		1 063 155	55 522 586
Oil	†616 133 bbl.	123 227	2 372 949
Miscellaneous		91 029	27 308 788
	Total	2 371 628	$96 048 996
PORT OF TACOMA.			
Lumber	268 221 170 ft. b. m.	404 427	$3 087 293
Grain		137 412	4 056 344
Flour		125 589	5 084 121
Fodder		19 747	271 730
Fruit		4 918	497 400
Coal		49 565	151 292
Oil	303 890 bbl.	60 778	528 728
Building material		163 022	1 008 541
Merchandise		170 473	31 290 942
Miscellaneous		163 275	15 339 051
	Total	1 299 206	$61 315 442
PUGET SOUND, SEATTLE AND TACOMA EXCLUDED.			
Lumber	1 196 745 204 ft. b. m.	1 686 207	$10 503 009
Grain		23 489	626 875
Flour		4 283	133 621
Fodder		12 705	234 012
Fruit		9 021	542 393
Coal		16 431	67 793
Oil	15 410 bbl.	3 142	24 527
Building material		178 230	537 253
Merchandise		136 511	10 084 509
Miscellaneous		174 870	6 138 503
	Total	2 244 889	$28 892 495

* Compiled from the Harbor Master's Report.
† Computed in part.

Vancouver, B. C.—The chief Canadian port, Vancouver, B. C., is in the vicinity of the mouth of the Great Fraser River, though actually not in the valley of that stream. More fortunate than Portland, Vancouver has been able to flank a troublesome river with its floods and shoals while still taking advantage of its valley on its rail route through the mountains. The port has been built in a land-locked bay called Burrard Inlet, surrounded on nearly all sides by towering hills which give the location a most beautiful setting. The area of the entire inlet inside the entrance is 23.6 miles, with a shore line of upward of 50 miles; but the port proper, as at present developed, utilizes only a small fraction of this. The shores are mainly of rock formation; the tidal range is 11 ft. mean and 16.4 ft. extreme. There is no ice, and the harbor is perfectly sheltered and has good anchorage.

The port facilities, as developed so far, are mainly on the south side of the east 5.5 miles of the Inlet. The portion of the Inlet comprising the harbor is about 1⅓ miles in width, connecting with the Gulf of Georgia on the west and the remainder of the Inlet on the east by two narrow channels designated First and Second Narrows, respectively. These Narrows were formed by the deposit of gravel, sand, and small boulders brought down by mountain streams on the north side of the Inlet and deposited in the shape of extensive bars. The Narrows are deep, but only about 500 ft. in width. Navigation through them is seriously handicapped by strong tidal currents, which, at the First Narrows, reach as high as 8 knots per hour at spring tide, and are such a menace to navigation that the Dominion Government has undertaken to widen the entrance by dredging to 1 200 ft. The improvement will not affect the current perceptibly, but will give more leeway for the navigation of ships. In most portions of the harbor the tidal currents interfere seriously with the berthing of ships.

New Westminster.—Ten miles south of Vancouver on the Fraser River, and 16 miles above its mouth, is the Town of New Westminster, which is taking active steps to develop port facilities for deep-sea shipping. The Fraser is a large river, not unlike the Columbia, but with a channel less difficult to improve and maintain. The ordinary tidal range at the port is only 5 ft., but the range between high and low water in the river is 14 ft. Reference will be made in another place to the steps being taken to develop this port.

Victoria.—This has been an important port in the past, and is so still, though its commercial business is not large. It is the chief port of Vancouver Island and on it is located the capital of British Columbia. The commercial port is on a small land-locked inlet with bald rocky shores. A few miles to the west is the celebrated Esquimault, an excellent port, strongly defended, and, prior to 1905, the headquarters of the British North Pacific Naval Squadron.

Prince Rupert.—This harbor is on an island within a well-sheltered inlet on the "inside passage" to Alaska, a little north of the mouth of Skeena River, the valley of which stream is the route of the western division of the railroad. The area of the entire inlet in front of and back of the island is about 27 sq. miles, and that of the shore line is about 80 miles. The tidal range is large, the maximum being 27 ft. and the mean about 14. The harbor is perfectly sheltered and is free from ice.

The port is very new, and its development has only begun. It now has about 2 700 ft. of berthing space, of which 600 ft. belongs to the Provincial Government and the rest to the Grand Trunk Pacific. It has a sectional dry dock 600 ft. long. The newness of the port and the fact that rail connection east is not yet opened, give little as yet for comparison with other ports. Its record lies all in the future.

The promoters of this important terminal enterprise adopted the almost unique course in civic growth of laying out their city before its occupancy by settlers began. It has been scientifically planned by artists of national reputation, and, if the plans are followed, it will be spared the disfiguration characteristic of most American cities.

III.—Notable Engineering Works.

The development of the ports of the Pacific has evolved a great range of engineering problems, some of which are of unusual magnitude, difficulty, and cost. A brief reference to the more important will be given. They include:

(1) Rock removal.
(2) Breakwater construction.
(3) Jetty construction.
(4) Lock construction.
(5) Dredging.
(6) Sea-wall and pier construction.

Rock Removal.—This class of work has been confined mainly to San Francisco Harbor and to ports north of the Strait of Juan de Fuca. In San Francisco the removals have been almost entirely of isolated rocks, some of which did not originally come to the surface. Another example of sunken rock removal is that in Roche Harbor, San Juan Island, Washington. Ledge rock removal has been and is being extensively carried on in British Columbia ports.

The various rock removals in the vicinity of San Francisco include the following: Blossom Rock, directly in the channel leading from Golden Gate into the harbor; Rincon Rock, now within the pier-head line and covered with a wharf; Arch Rock; the two Shag rocks; Centissima Rock, in the Bonita Channel just outside the Golden Gate; and Noonday Rock, 33 miles at sea opposite the harbor entrance. Of these removals, Blossom Rock was most important. The method of removal originated with General Alexander of the U. S. Corps of Engineers, and the plans were very carefully worked out. They included the building of a coffer-dam on top of the rock, which rose to within 6 ft. of low water, unwatering the coffer-dam, sinking a shaft into the bed of the rock, and then excavating an immense chamber into which was to be placed the explosive. It was expected that the strong tidal currents would remove most of the shattered rock, and that the remainder would be scraped into deep water. The contractor sank and securely fastened to the rock a timber crib which served the double purpose of a platform and a protection to an iron tube 6 ft. in diameter and 14 ft. high, which was placed through the center of the crib. A water-tight connection was effected with the rock by the use of a liberal quantity of cement. The tube was then unwatered and a shaft sunk to Reference 30 ft. below low water. During the progress of sinking the shaft so much leakage occurred through the rock that it was found expedient at two different times to telescope smaller tubes into the shaft and secure water-tightness by cementing the annular space between the rock sides of the shaft and the tubes. At a depth of 14.5 ft. into the rock the leakage became nominal, and the chamber was then easily excavated. The floor of the chamber was at Reference 31.5 ft. below low tide, the axes being 135 and 55 ft., respectively. The roof conformed to the general shape of the surface of the rock and had a thickness varying between 14 and 18 ft. Four stone pillars were left at the shaft, and timber supports were distributed throughout the chamber.

The explosive used was nitrate of soda, of which 4 300 lb. was placed in 38 wooden, ale barrels and in 7 old, boiler-iron tanks. The barrels were distributed around the edge of the chamber, and the tanks were scattered at about equal intervals on the floor. The blast took place on April 23d, 1870. The explosion was a pronounced success, but the chamber had not been made large enough to engulf the broken-up roof, and the hope that the current would remove the débris was not realized. The subsequent removal of débris, by scraping the blasting, cost as much as all the previous operations, up to and including the blast.

A contract for removing an additional 6 ft. from Blossom Rock in order to give a 30-ft. depth at low water, was let in November, 1902, for $45 142. The quantity of rock to be moved was 4 630 cu. yd. The contractor drilled a few holes, but abandoned the process for surface blasting and dredging the broken-up rock into deep water. The official reports do not give the reasons for the adoption of this method, but it may be conjectured that at least one reason was the shattered state of the rock due to former operations.

On January 2d, 1863, the clipper ship, *Noonday,* was wrecked on an uncharted rock in the Pacific opposite Golden Gate, and a demand arose for its removal. A search for the rock was made by the U. S. Coast Survey, and a pinnacle 5 ft. square on top, with 19½ ft. of water over it at low tide, was found and buoyed. In 1874 a contract was awarded to Mr. Edward Moore, of Portland, Me., for removing the menace to navigation, at an upset price of $20 000. Being in the open sea and subject to adverse weather conditions, the work entailed more hazard than similar work in a sheltered harbor. At first it was proposed to sink an encircling chain to a depth of 50 or 55 ft., at low tide, to which were to be attached packages of nitro-glycerine, but when operations were commenced the diver discovered below the required depth a cavity 10 ft. deep in one side of the rock. The plan was altered immediately to put the entire charge of explosive in the cavity. The charge was fired May 7th, 1875, and proved an entire success.

The method of subaqueous rock removal known as "bulldozing" apparently had its origin in San Francisco. It consists simply in placing high explosives in contact with rock surface and exploding it there without drilling or tamping. The shattered surface is then scraped away and the process repeated. Although the use of explosives

by this method is excessive, the expense of drilling is saved. The results, so far, however, do not seem to commend the method on the score of either efficiency or cost. In 1907-08 a submerged rock at the entrance to Roche Harbor, Puget Sound, was removed by this method. The volume of the portion to be removed was 1 500 cu. yd., and the contract price was $23 460; but the work probably cost the contractor more than this. The official Government report says that "the method, as applied in this case, did not prove satisfactory, and the contractors state that if they had the work to do over they would drill the rock."

In the work in San Francisco Bay, where the masses to be removed were considerable, the contract price per cubic yard was less than $10; as, for example, the work on Rincon Rock, before it was abandoned, cost $9.06 per cu. yd. for the removal of 4 357 cu. yd.; and the cost of removing 32 000 cu. yd. on the Shag and Arch Rocks was $7.93 per cu. yd.

The subaqueous basaltic rock excavation at the eastern end of Victoria Harbor was performed by the drilling method. The single drill was mounted on a platform float that could be raised clear of high tide by steam power spuds. Steam was furnished from boilers on a barge. Thirty-two holes were drilled at one setting of the raised platform. The holes were 2.5 in. in diameter, drilled to a depth of 2.5 ft. below grade (20 ft. at low water), and spaced 3 by 3 ft. The actual cost for drilling and blasting only was $6.17 per cu. yd., without interest, depreciation, or plant renewal.

The blasting in Vancouver Harbor will be a more delicate operation, as it will be carried on in close proximity to constructed piers. Holes 3 in. in diameter, spaced 5 by 5½ ft. centers, will be drilled to a depth of 3 ft. below the required grade line (36 ft. at low tide), and shot off in series. It is expected that 1½ lb. of dynamite per foot of hole will be used. The estimated cost for drilling and blasting is $5.60 per cu. yd. Dredging the broken rock will approximate an additional $1 per cu. yd. The Vancouver rock is a sloping sandstone ledge overlaid with from 3 to 7 ft. of gravel, hardpan, and disintegrated sand rock.

Breakwater Construction.—As an example of breakwater construction strictly speaking—that is, work done for the primary purpose of protecting anchorage areas or harbor entrances from ocean storms—the San Pedro Breakwater at Port Los Angeles is the most important

on the Coast. As finally constructed, it has a length of 11 000 ft. and extends first on a curve and then in a straight line easterly from Point Fermin, thus giving shelter against south and southwest exposure and in particular completely protecting the entrance to the inner harbor which is behind the inner end of the structure. The breakwater is built entirely of rock, except a concrete block 40 ft. square and 20 ft. high at the outer end. The base of this block is 3 ft. below the low-water plane. The construction of the breakwater was effected from a double trestle carrying tracks of standard gauge. The substructure was built to the elevation of low water and consists of a central portion of stone weighing 1 ton or less, with larger stones on the side slopes. The superstructure is 38 ft. wide at the bottom and 20 ft. wide at the top (14 ft. above low water), with stepped sides of huge blocks of stone placed by cranes. On the ocean side the stones weigh 16 000 lb. or more; on the harbor side, 6 000 lb. or more. The space between the sides is compactly filled with rock of all sizes, and the top is finished with large stones. The side blocks have been placed with such care that they seldom depart more than 6 in. from exact position.

Jetty Construction.—As used in this connection, the term, jetty, applies to a structure designed to serve the primary purpose of concentrating and directing ocean currents to develop scouring effects over shoal bars, and the secondary purpose of protecting such crossing from storms, as far as their location for the first purpose will permit. They are, therefore, a combination of training dike and breakwater. There are four prominent examples on that portion of the Pacific Coast considered in this paper, all of which are north of San Francisco, namely, Humboldt Bay, Coos Bay, the Columbia River, and Grays Harbor. There are also numerous minor examples, as at San Diego, Los Angeles, and Oakland. The principles involved are essentially the same in all, though varied in application, of course, to suit the conditions of each case. The Columbia Jetty far exceeds any of the others in importance and in the magnitude of the work, and the description will be limited to this one example.

As originally projected, the jetty extends in a west-by-northwest direction 4½ miles from the east side of Point Adams across Clatsop Spit and out to deep water. The jetty was to be built of large size stone on a brush mattress and raised to the level of low tide. Later,

the project was changed to raise it to high tide, and four groins were to be built from the north side to arrest scour. Work was begun in 1885 under an estimate of $3 710 000, and was completed ten years later at half the estimate. The depth over the bar had increased, in the meanwhile, from 21 to 31 ft., and the work seemed to have accomplished its purpose perfectly.

Unfortunately, the bar began rebuilding itself, the depths shoaled, and several years later there was only the original depth of 21 ft., and it seemed to be necessary to start all over. A new project was adopted, extending the jetty 3 miles farther, or somewhat beyond the longitude of Cape Disappointment on the north shore. The work was to be supplemented by dredging over the bar. This extension is now nearing completion and has resulted in a present depth of from 25 to 27 ft. A permanent depth of 30 ft. is sought.

This work, it is scarcely necessary to remark, is probably the largest and most difficult of its kind ever attempted, and has evolved some of the most ingenious and elaborate methods for overcoming difficulties in the whole range of engineering. The two great obstacles to be overcome in the execution of the work are the storms and the teredo, and it is the fact of the storm that gives the teredo its "inning." The roughness of the sea precludes the use of barges for dumping rock, thus necessitating a trestle, and the trestle piling is the particular delight of the teredo, which puts it out of commission in from 10 to 20 months. The rebuilding of the trestle on the rock of the jetty, which has been repeatedly necessary, and the methods resorted to in order to arrest the work of the teredo, have taxed the resources of the engineers to the utmost.

The jetty itself, as now being built, consists of a bed course of small rock as a substitute for brush mattress, it being found impracticable to use the mattress beyond the shoal depths of Clatsop Spit. On the foundation so provided heavier rock is deposited, and the whole is covered on the seaward slope with very heavy rock ranging in weight from 6 to 16 tons per piece. Even these enormous masses are displaced by the storms, and the embankment is progressively shaken down during each winter season.

The long continuance of this work has evolved a very efficient system for its management—one which it would be difficult perhaps to improve on, because every feature is the result of actual experience.

The pile-driver is a study in itself. The support of the pile trestle on its bed of huge boulders, the arrangement of the movable car beds and of the cars themselves for the quick handling of the rock, the systematic organization of the work, which moves with the regularity of clockwork, and, finally, the rational division of the work between the contract and day-labor systems, assigning to each the share it is best adapted to carry out—all combine to make this great work well worthy the attention of the engineers of the United States.*

Lock Construction.—Locks are an important adjunct of port development in English ports, where they are applied to overcome the drawback of excessive tidal fluctuation. They are nowhere so used on the American Continent, but they are about to be used in the Port of Seattle for the purpose of effecting a very large extension of the harbor, as previously explained. The lock which separates the main harbor from the Sound is located at the outlet of Salmon Bay, 1 mile inland from deep water, at a sheltered and easily defended point called the Narrows.

The lock is really a double structure, comprising a large chamber for heavy shipping, log rafts, etc., and a small chamber for the use of the mosquito fleet. It is certain that small-boat traffic through the canal will be large, and the plans provide a small, quick-operating lock for its use so as to avoid the slower and more expensive operations of the larger lock. It is considered that this provision will prove one of the most useful features of the canal. A similar arrangement is provided on the Manchester Ship Canal, in England. It seems to the writer that if the Panama Canal plans are open to any important criticism it is in not providing a flight of smaller locks. To operate those immense locks for the passage of small boats cannot fail to prove a great waste of energy and time and possibly an interference with traffic.

Returning to the Lake Washington Canal lock: a description of that structure, which is now in process of erection, will be given. The larger lock will be 825 ft. long and 80 ft. wide, or large enough to admit without breaking the large log rafts which are certain to constitute an important part of the traffic. The sill of the lower gate will be at Reference 25 ft. below extreme low tide, 29 ft. below

* Those who desire a detailed description are referred to an article on page 109 of *Engineering News* of July 30th, 1908.

mean lower low tide, and 35.67 ft. below mean sea level. The sill to the upper gates will be 36 ft. below the low-water level of the canal and 38 ft. below the high-water level. The lock will be supplied with intermediate gates which may be used to divide the chamber into lengths of 350 and 475 ft., respectively, for convenience and economy of water in the passage of shorter vessels. There will be guard gates at the two entrances, and a movable dam at the upper end for emergencies arising from accidents. The small lock will be built on the south side of the large one, and will be 150 ft. by 30 ft. and 16 ft. deep over the miter sills.

The lock walls will be of mass concrete, gravity section, straight-front face, with a toe projecting into the lock chamber, and stepped back. The filling culverts will be of large elliptical section, running the full length of both walls, and connecting with the chamber by numerous side culverts at right angles to the axis of the lock. The controlling culvert gates will be of the Stoney type in the large lock and of some form of cylindrical balanced valves in the small one. It is expected to fill or empty the chamber of the big lock in 8 min.

The lock gates will be of steel, with parallel straight faces except near the front and back edges. All gates will be provided with air chambers to relieve the strain on anchorages and to permit of floating the guard gates out of position for repairs and painting. The main gates will be reached by closing the guard gates and pumping out the chamber.

The most interesting detail of the lock, and the one on which there will be most divergence of opinion, is the lock floor. The ever-recurring debatable question in lock construction arises here—shall the floor be made strong enough to resist an upward hydrostatic pressure, or shall it be a mere pavement to protect the underlying material from scour, reliance being placed on the cut-off walls to prevent leakage? The lock floor is not yet determined on, the decision being held in abeyance until an examination can be made of the foundation material, when the lock coffer-dam is unwatered.

The lock when completed will be exceeded in size, on the American Continent, by the Panama Lock only; it will be on a par with the new Sault Lock, greater in some particulars, but less in others, and will entail the solution of more difficult engineering problems than the one at the Sault. It will rank with the greatest European locks, and take

its place in shipping and engineering circles as one of the very prominent structures of its class.

The principal data relating to the canal are as follows:

Extreme low tide in Puget Sound	Reference	0
Mean lower low tide, " "	"	4
Mean low tide, " "	"	6.89
Mean sea level, " "	"	10.67
Mean high tide, " "	"	14.50
Mean higher high tide, " "	"	15.23
Extreme high tide, " "	"	18
Present level of Lake Union	"	25
" low-water level in Lake Washington	"	33
" high-water level in Lake Washington	"	38
Proposed low-water level in canal	"	24
" high- " " " "	"	26
Length of large lock between quoins	825	ft.
Width " " " " "	80	"
Depth on lower sill at extreme low tide	25	"
" " " " " mean sea level	35.67	"
" " upper " " low water	36.00	"
" " " " " high "	38.00	"
Lift of lock from extreme low tide to high water	26.00	"
" " " " " high " " low "	6.00	"
Depth of water in main body of Lake Union	40.00	"
" " " " Lake Washington	40.00 to 200.00	"
Length of canal from deep water in Puget Sound to deep water in Lake Washington	8.00	miles

Dredging.—Pacific Coast ports are no exception to the almost universal need of dredging operations to secure full navigable depths in channels and docks. Seattle and Tacoma come nearest to being exceptions, but even in these ports waterways have been created artificially by dredging. At Seattle, for example, a great deal of work has been done in reclaiming the tide flats of the Duwamish Delta, and creating artificial waterways there, while now extensive operations are in progress in excavating the Duwamish Waterway and the Lake

Washington Canal. None of these operations was necessary, however, in the main harbor, which everywhere had ample depth. At the other extreme is Los Angeles, where the harbor itself is being built almost entirely from excavation. Differing from either of these is Portland, where the channel of approach to the harbor requires very extensive and continuous dredging, not only for original deepening, but for maintenance of depth. San Diego, Oakland, San Pablo Bay, Grays Harbor, and several of the ports in Puget Sound, have been thus improved by excavation. As a rule, this work has been done with Government appropriations, though in some ports, such as Portland, and, more recently, Los Angeles and Seattle, local funds are being contributed to the work. Now that the reclamation of tide flats is becoming so profitable an enterprise, dredging will be resorted to more than ever, the operation serving the double purpose of excavating slips and channels and filling the abutting lands.

The plant used in this work embraces every practical variety of dredging machinery, from the heaviest sea-going hopper or suction dredge to ordinary steam shovels loaded on scows. The Pacific Coast dredging fleet, taken as a whole, both publicly and privately owned, is a formidable one. In the more localized operations electricity is sometimes used as the motive power. Oil is used extensively as fuel. All varieties of excavation are encountered, from the lightest alluvium to indurated clays that border closely on soft rock. In California great use is made of the clam-shell dredge, with enormously long booms, designed for levee building on the Lower Sacramento and San Joaquin Rivers, and on some tributaries of the larger stream great hydraulic dredges are used to glean out the particles of gold which the improvident miner wasted as he sluiced down the auriferous hillsides a generation and more ago.

It is needless to describe in any detail these dredging operations, for they present nothing sufficiently characteristic to mark them from those with which engineers are familiar in the East. It will be of interest, however, to refer briefly to two of the dredges. The latest sea-going hopper dredge built by the United States Government is the *Chinook,* for use on the Columbia River Bar. It is a converted transport, 445 ft. long, of 5 590 gross tonnage. The transformation was made in 1902, but many alterations and repairs have been made since that time. The dredge has two 20-in. pumps and two

bins of a total capacity of 3 600 cu. yd. Its performance during the fiscal year ending June 30th, 1911, was:

Days worked	160
Cubic yards excavated	722 431
Total operating cost	$58 943.15
Cost per cubic yard	$0.08159

Probably the most effective dredging plant for all purposes on the Pacific Coast is the *Frühling,* built in Germany in 1906 at a cost of $275 000, exclusive of duty and cost of steaming to British Columbia. It is the property of the Department of Public Works, of Canada, and operates on the Fraser River. It is of the "Frühling" scraper suction type, elevating material by a pair of 16-in. centrifugal pumps through pipes and delivering to hoppers inside the hull. It is a sea-going dredge, of steel construction throughout, and is self-propelled by twin screws. Its length between perpendiculars is 187 ft.; the extreme beam is 34.5 ft., the draft, loaded, about 14 ft., and the hopper capacity at this draft, 800 cu. yd. Its performance from April 1st, 1910, to March 31st, 1911, was as follows:

Hours worked	862.5
Cubic yards excavated	924 800
Average cubic yards per hour	1 072
Average cubic yards per day of 9 hours, including all delays	3 000
Maintenance cost	$37 948.10
Repairs cost	8 835.58
Total cost	46 783.68
Cost per cubic yard	0.0505

Sea-Wall and Pier Construction.—As stated elsewhere in this paper, there has been very little massive sea-wall construction in the ports of this Coast, San Francisco being the only considerable exception. The earlier type of cross-section adopted there consisted of a loose rock core faced with heavy stone paving. More recently, reinforced concrete on a rubble and pile foundation is being used. There have been some attempts to build vertical concrete walls in Seattle Harbor, but the excessive cost and the injurious effect of the sea-water on the concrete have not led to their repetition. Where the tidal range

is as great as it is at Seattle, a sea wall has to stand above the foundation fully 60 ft., and the cost of a wall built on any adequate plan is almost certain to exceed $300 per lin. ft. This condition has led to various schemes for supporting the land face without the cost of a wall. Creosote or concrete piling, concrete piers, or cylinders supporting a platform back for a distance of from 30 to 60 ft., afford an opportunity to carry the slope up at an angle which can be made stable at moderate expense with brush or rock paving or a combination of both. Much thought is now being given to this problem, and a type of construction will doubtless be evolved which will be a fair compromise between cost and durability.

The same difficulties enter the problem of pier construction. The method heretofore in force along the Coast has been mainly that of wooden piling and timber superstructures—in fact, the simplest form of practical construction. Latterly, there has been a strong wave of sentiment for "permanent" work, which means reinforced concrete substructures and fire-proof superstructures. Conservative considerations, however, are going to place a strict limit on the application of this method, and timber construction, with creosoted piling, where subject to the ravages of the teredo, will continue to be the rule. Any extensive adaptation of the costly masonry construction of European ports is not to be expected.

The serious problem, of course, is that of the substructure. Ordinary piling will not last a year in these waters. Creosoted piling, if of superior quality, is safe for fifteen years; but the lack of stability in very deep water, where depth necessary for draft is augmented by a high tidal range; the great cost of replacement, requiring, as it often does, the removal of shed roofs and the suspension of business; the increased danger from incendiarism, rats, theft, etc., and draft in conflagration, all due to the open work underneath, are very serious drawbacks which go far to offset the advantages of less cost. The most practical compromise with timber construction would seem to be creosoted piling under wharf platforms with protected slopes to bulkheads on line with the shed walls, thus giving solid fills under the sheds. Concrete piling or concrete cylindrical piers, such as are used in the splendid new Government piers at Fort Mason, San Francisco Bay, are, of course, more to be desired for this exterior work. A brief description will be given of two docks, one a fine example of

typical timber construction and the other an original plan of solving the difficulties referred to.

The Grand Trunk Pacific Dock, at the foot of Madison Street, Seattle, is a recent structure. It is 625 ft. long, 128 ft. wide, and is supported entirely on creosoted piles, the length of the bearing piles varying from 55 to 90 ft., with bracing piles as long as 110 ft. The bents are 10 ft. apart, and the piles in the bents are spaced from 6 ft. centers at the inner end to 3 ft. at the center. The capping and floor systems are of very thorough construction, and the dock appears to have thorough stability, notwithstanding the great depth of water in which it stands.

The main shed is 90 ft. wide, with a platform 8 ft. wide on the south side and 16 ft. on the north, the latter carrying a railroad track. There are five adjustable slips on each side and one at the outer end worked by worm gear. The fender piles are of tight-bark fir, and are driven two at every bent and fastened to 12 by 12-in. stringers with $\frac{3}{4}$-in. straps. At the outer end they are spaced 3 ft. apart, and clusters of seven piles are driven at the outer corners. The timber used in construction was 241 775 lin. ft. of creosoted piling, 71 000 ft. B. M. of creosoted bracing, and 1 113 000 ft. B. M. of other lumber. In its finish, offices, equipment, and general lay-out, it is the best example of dock construction on Puget Sound.

The second of the docks selected for a brief description here is the Great Northern Dock, at Vancouver, B. C. It was designed by A. W. Münster, M. Am. Soc. C. E. The pier is 450 ft. long and 293 ft. wide, with a berthing space of 30-ft. depth at low water on each side, and with two sheds, 403 by 102 ft. Between the buildings is a space 41 ft. wide. The distinctive feature of the structure is a cantilever platform supported on cylindrical concrete piers, which themselves rest on a sandstone ledge. These piers are 4.5 ft. in diameter, with a base enlarged to 10 ft. The foundation is carried down into the hardpan or sandstone ledge to a minimum depth of 3 ft. below the bottom of the slip. The piers will have an average height of 52 ft. On top of them will be placed a longitudinal concrete girder, 3 ft. wide and about 7 ft. in depth, which will carry the transverse girders forming the immediate support of the floor slab, which are placed 12.5 ft. center to center. The transverse girders are from 2 to 3 ft. wide, 6 ft. deep, and 51 ft. long. They will project 16.5 ft. beyond the line of

the piers and, acting as a cantilever, will carry corresponding parts of the front platform. The inner end of these girders will rest on a line of 16-in. concrete piles driven to refusal and stayed in position before the rock embankment is placed. The floor slab is to be 7 in. thick. The concrete platform and beams have been designed for a distributed load of 500 lb. per sq. ft. The railroad track running the length of the platform is placed centrally over the longitudinal girders. The reinforced concrete construction supporting the track is proportioned to carry a 100-ton switch engine. Tie-rods encased in concrete will anchor the concrete platform firmly to the embankment.

The warehouse buildings are of wooden construction covered with galvanized iron. The roof is to be carried on trusses spanning the whole width of the building, leaving an unobstructed floor space. The floor inside of the building will be paved with creosoted blocks on a concrete base. The concrete slabs on the outside platform are to be covered with an asphalt wearing surface. The cost of the completed dock and buildings will be approximately $1.50 per sq. ft.

IV.—Administrative Systems.

Public Ownership and Control.—In discussing the different systems of administrative control which obtain in the ports of the Pacific Coast, it is necessary to take cognizance of the present strong trend of public thought in favor of public ownership and control of port facilities. It would be interesting to examine the reasons which underlie this movement, but it must suffice for the present purpose to say that they are essentially the same as those that have substituted the free public highway for the toll road of former times. For the wharf is in reality the transfer point between a free public highway on the land and a free public highway on the water, and there is no more reason why this point of transfer should be made a subject of private monopoly than either highway. The transfer may not indeed be made wholly free, and the user may properly pay the actual cost of use, just as a user of the public highways pays another for hauling himself and goods along it when he does not perform that service himself. Moreover, a public wharf being built and maintained through the agency of only a limited portion of the community actually using it, the cost cannot be equitably charged to that portion alone, as it would be if absolutely free of tolls and maintained

solely by taxation. A strictly free port is not, therefore, one of the things to be expected or desired, and is not likely to be realized except as a result of cut-throat competition between rival ports. The writer will not go further into this extremely important matter of port control than to repeat a summary of the principles involved as stated by him in a recently published article.*

"The true policy for a great port to pursue in these matters would, therefore, seem in the light of present-day advanced practice, to be essentially as follows:

"The wharves with the buildings and appurtenances directly connected therewith should be built and owned by the port itself, and should be operated either directly or by short-term assignments in such way as to eliminate entirely the element of private monopoly, and to bring the cost of service to the lowest point consistent with maintenance, operation, and gradual recoupment of original outlay. Even when this return cannot be fully realized and the utilities have to be supported in part by taxation, public ownership may still be vastly advantageous through indirect benefits, just as are public highways which yield no direct revenue whatever.

"The system of railway tracks serving the wharves should be consolidated in the larger ports, under a single unified management, separate and distinct from the tributary railway systems, except as they form the connecting link between such railways and the wharves. These service tracks may be managed either by a private terminal company, subject to port supervision, or by the port authorities themselves. The latter method has definite advantages.

"So far as a port may directly or indirectly foster industrial development in connection with its own utilities, it should do so upon terms which will attract private capital and insure it a sufficient lease of life to enable it to work out its plans and enjoy the fruition thereof. The case is essentially different from that of public utilities, and so far as a port may be drawn into such enterprises it must necessarily be upon a different basis. There is no question that many industrial enterprises have so close a connection with water and rail transfer facilities as to make them quasi-public enterprises. While they are not of a character which a port will ordinarily take up as an original proposition, they are entitled to every practicable measure of cooperation and assistance which the port may give, and are to be considered as an essential element in its successful development."

In many European ports there is a close approach to this ideal, though with wide variations under different governments and customs.

* *Railway and Marine News*, Seattle, Wash., May 1st, 1912.

In America the ultra-democratic ideal of *laissez faire* and non-interference by Government led to an almost universal custom of private ownership of harbor frontages. Astute business foresight was not slow to appreciate the opportunity thus offered, and the important points of every important harbor, almost in its infancy, became private monopolies. When the railroad came, this evil assumed an aggravated form, and it would be a threadbare story to describe the extensive appropriation of water frontage by the railroads, particularly in the newer sections of the country.

As time has passed and the public has been brought face to face with a situation resulting from its own earlier indifference, a change in sentiment has taken place and is just now in full tide of progress. The people are determined to recover the birthright which they bartered away for a paltry mess of pottage. It is no paltry ransom, however, that they are being compelled to pay. Untold millions are being paid and will continue to be paid for what should never have been parted with at all; but the people are determined, cost what it may, to regain control of their water fronts. Private interests resist—it is the loss of a rich monopoly—but they cannot prevail, and the force of public sentiment should be brought to bear to make the struggle as brief as possible.

San Francisco.—Among the ports of the Pacific Coast, San Francisco is the one bright exception in the past to the rule of private ownership. The principle of public ownership and control has, indeed, been adopted in all three of the Coast States; but it has failed utterly in application except in the Port of San Francisco. A significant proof of the overshadowing importance of this port in the past history of the State of California is seen in the fact that early legislation on port matters was directed almost exclusively to San Francisco, and a system of government was established there which was not applied to any other port of the State. The statute which created the Board of State Harbor Commissioners (note, it is a State tribunal, perfectly general in its title) limits its jurisdiction to "possession and control of that portion of the Bay of San Francisco" (description by metes and bounds takes up three pages in the printed statute), extending in general terms from the Federal Reservation (the Presidio) near Golden Gate around into the Bay to the line between San Francisco and San Mateo Counties. Though a State body, its

activities are purely local and pertain exclusively to a portion of the water-front of a single city of the State. Its officers are appointed by the Governor and confirmed by the Senate, and are a regular part of the political machinery of the State. The first Board of Commissioners was appointed in 1863, and the Board has been in existence, therefore, for very nearly half a century.

Whatever may have been the failings of this body—and it has been accused of many things, particularly of being a subservient political organization—the fact that, through all these years, in a State where railroad influence was supreme, it has maintained the integrity of the water-front of San Francisco and has kept it in public control, must stand to its everlasting credit. To-day San Francisco, New Orleans, and Montreal are the only American ports in which the principle of public ownership is practically applied on a large scale, and though the system in force in San Francisco is not as perfect as that in New Orleans, it is a vast improvement over that which prevails generally in the ports of this country.

The vicissitudes of economic progress, however, have at last made the system of State control antiquated, and San Francisco is now ready to forego the pupilage of the State and take her future into her own hands. She wants home rule. She notes with anxious eye the prodigious strides of Los Angeles, San Diego, and particularly the cities across her own Bay. These ports are helping themselves, except that they are receiving from the State grants of tide lands amounting to many thousands of acres. They are organizing on the basis of their own needs. They issue their own bonds. They prepare their own plans. They fight their own battles. They are not tied down to legislative sanction for the measures they desire to adopt, and do not have to go to the electorate of the whole State for financial aid. They may be no longer willing to help pay San Francisco's bills, and it behooves San Francisco to get authority to pay them herself. She is waking up to the situation and is now earnestly agitating for complete home rule of her port. Undoubtedly it will come, and with it a more energetic and efficient working out of her destiny.

The small importance of the east side of the Bay in the early years made it an easy prey to private interests. Although the water-front of Oakland was granted to the town by the State as far back

as 1852, it immediately passed into private hands on a long-term lease, and finally fell into the control of the Southern Pacific, from which it has only recently been wrested by strenuous litigation. The port is fully alive to its opportunities, and the course of development from now on will unquestionably be in the direction of public ownership and control. As yet there is no port organization separate and distinct from the City Government. In time the east-side towns undoubtedly will consolidate, and a comprehensive port organization will be developed. Whether such an organization for the whole of San Francisco Bay will ever be adopted is doubtful. The jealousies of the two sides are too strong to make union practicable at present; but to the onlooker from the outside, a port commission for the whole of San Francisco Bay, with unified development of the entire port, would seem to be the best plan.

Los Angeles.—In Los Angeles there is no separate port organization distinct from the City, but the policy of public ownership is making great headway. Probably the bitterest struggle ever waged to rid a port of the incubus of private monopoly of its water-front is that of Los Angeles against the allied Southern Pacific and other interests. Of the final triumph of the people's cause there is no longer any doubt, though the delay and expense of litigation are a severe strain on the patience of the community. The administration of the port is divided on indefinite lines between a board of public works and a board of harbor commissioners. The method, of course, is unbusinesslike and has resulted, as might have been foreseen, in conflicts of authority which have seriously disturbed the orderly procedure of the work of port development. One of the first problems which Los Angeles should solve is the working out of an efficient system of port administration.

San Diego.—San Diego furnishes an interesting example of the prevalence of home-rule sentiment in Californian ports. In 1910 the State voted $1 500 000 in bonds for the development of the harbor; but the people preferred home rule, and the Legislature annulled the grant and ceded the tide lands within the city limits to the City on condition that the latter shall expend $1 000 000 in port development along specified lines and within a specified time. There is no separate port organization in San Diego, and the development work is carried out as an adjunct of the City Administration.

Portland.—The Oregon system of port management is admittedly not a satisfactory one, and, as exemplified by its application to the principal port of the State, it is in no sense an efficient one. It does not embrace any comprehensive scheme of port development and control, and, until quite recently, it contemplated nothing in the line of public ownership and control of dock facilities; it left the whole matter to private enterprise. The river front was originally deeded to the City, but has passed almost entirely into the hands of private interests—mainly that of the railroads. So strong was the hold of this system that, when the "Port of Portland" was created twenty years ago, its purpose was not at all to provide port facilities, much less acquire or control those in existence, but solely to secure and maintain a 25-ft. channel to the sea. It was simply an ally of the Government in its regular work of channel improvement. Certain specific functions have since been added, such as control of pilotage charges and the building of a floating dry dock; but in its essential purpose it is still an agency for channel improvement only. Its name, therefore, is a misnomer, and, when public sentiment became aroused recently to the necessity for some actual measures in port development, a new body was organized—the Portland Dock Commission—to take up this particular line of work. A bond issue of $2 500 000 was authorized, and plans are actively in progress for the construction of a system of public docks. On the face of it, the whole arrangement appears to be of an unsatisfactory and piecemeal order. Neither the original nor the new commission, nor even both together, has anything like a comprehensive jurisdiction; but their work has to be supplemented by co-operation with the City, and thus there is a sort of three-cornered jurisdiction of the port. It is scarcely to be expected that such a system can be permanently satisfactory. In all probability there will result a consolidated port organization, which will comprehend the present divided jurisdictions and proceed systematically with a unified upbuilding and administration of the port.

The Washington State Law.—The State of Washington has recently taken the most advanced position of any of the Pacific Coast States in this important matter of port administration. By Act of March 14th, 1911, it provided a system of comprehensive port development and control under an organization for that exclusive purpose. The organization is styled a Port Commission, and is composed of

three members elected for terms of one, two, and three years, respectively, in the first instance, and one Commissioner each year thereafter for the three-year term. The powers of the Commission are very comprehensive, embracing the whole range of port development and a large measure of port control. Certain powers are still lacking for the legal enforcement of rules, etc., and there is doubt as to the extent of the Commission's power in promoting industrial enterprises. There are, however, certain crude provisions which need modification to make the statute run smoothly; but on the whole it is the most elaborate, comprehensive, and effective measure of the kind ever passed. It will take time for it to gain full recognition, for it strikes at the root of private monopoly in communities where a large element of the so-called "business" interests is immovably committed to such monopoly; but, if it ever has half a chance to demonstrate its efficiency, it will command popular approval. Two districts have already been organized—one in King County, in which Seattle is located, and one in Chehalis County, which embraces Grays Harbor. In the Seattle District the law has been put into effect with great vigor, and a programme of $6 000 000 has been adopted by popular vote.

The development of the Port of Seattle prior to the passage of this law is a typical example of the complete miscarriage of a great purpose. In no State in the Union was a broader view taken of public questions of this character in framing the Constitution than in Washington. It was the purpose of the State to hold the tide lands in perpetuity for the use of the people, and the first Board of Harbor Line Commissioners was in earnest and determined to abide by the purpose of the Constitution. It would have been a great thing for Seattle if it had prevailed; but the time was not ripe. Public sentiment did not support it. It robbed private interests of too many opportunities for speculation. The false cry of business necessity prevailed, and in one way and another the Constitutional injunction was completely nullified, and the tide lands, now worth many millions, passed wholly into private hands at almost nominal rates. Streets were vacated, and even the harbor area, the phantom relic of a beneficent intention, was so tied up to abutting lands as to become a private monopoly. To-day the Port Commission, in trying to recover enough of this wasted heritage to build its first piers, is asked to pay, in some instances, as high as $86 000 per acre.

British Columbian Ports.—The British Columbian ports, particularly Vancouver, are wholly private-ownership ports, and the water-front development is almost exclusively in the hands of the railroads. It is a striking contrast to the great Canadian port of Montreal, which is one of the most progressive public-ownership ports in the world. Vancouver is passing through the boom experience of most new localities in the West, and is willing to make almost any concession to private interests for what promises most for the immediate present. Not until after the present period of expansion has passed, and public thought, now absorbed in speculation activities, has time to take stock of the situation, is it likely that the problems of port control will be given very serious consideration.

Port Charges.—The diversity of administrative method among the Coast ports is particularly manifest in port charges. The widest difference prevails, and in only one port is there a definite and uniform system. San Francisco has a well-defined schedule of charges, and, whether the best adapted to the purpose or not, it is carefully worked out and serves its purpose very well. In most other ports of the Coast, rates are a matter of private arrangement, although in each there is generally a voluntary agreement among shippers as to what they shall be. They vary greatly, and even in the same port different rates are often applied according to the exigencies of the case. Thus, in Seattle, the basing rate on foreign cargoes is 25 cents per ton, and on coast and local cargoes 50 cents per ton. In San Francisco the whole matter is strictly regulated by a public body, and there is complete uniformity and absence of discrimination. Heretofore in Seattle there has been no such regulation except the ineffectual regulation of the State Commission, and there are frequent complaints of discrimination and irregularity. The two systems, moreover, are essentially different. In San Francisco there is a substantial dockage charge against the vessel; in Seattle there is no such charge, the entire charge being against the cargo. It is this custom which has given Seattle the reputation of being a "free port", but the freedom refers only to the vessel itself. The system is not an equitable one, and will have to be changed as soon as an agreement among the Sound ports can be reached. The San Francisco system is the more rational one in this respect. It comprises three fundamental charges, which have been defined officially as follows:

Dockage.—The charge made for vessels occupying berths at the wharves or moored in any slip or channel.

Tolls.—The charge made for merchandise passing over State premises.

Wharfage.—The charge made for leaving merchandise on State premises longer than the time specified in the rules.

In the Puget Sound ports there are no dockage charges unless a vessel is lying alongside a dock unengaged. The "handling" of the cargo corresponds to "tolls" in San Francisco, though it is less definite in its application and sometimes has no application at all.

Although the San Francisco system appeals to the layman as the more rational and just, the ship operator is positive in his preference for the Puget Sound system (if it can be called such) and holds that it is decidedly the cheaper port. As far as the vessel is concerned, this is true, but there is no great difference in amount where the total cost to ship and cargo is considered. Labor conditions in San Francisco make stevedoring higher than on Puget Sound, but this is a matter entirely apart from the port regulations in themselves. Labor unions may at any time force wages in other ports to the San Francisco scale, and, therefore, a correct comparison of systems should assume uniformity of wages.

In Los Angeles the charges are classified under the heads of dockage handling, and wharfage, "handling" corresponding to "tolls" in the San Francisco schedule. The rates in San Francisco are fixed by the Board of State Harbor Commissioners, supplemented by several statutory enactments. In Los Angeles they are fixed by the Board of Harbor Commissioners, a branch of the City Government. The City charter gives this Board the same legal powers which the State Board enjoys in San Francisco in regard to the enforcement of its rates and regulations, and gives it the power to punish violations thereof. In San Diego there appears as yet to be no system of local public control of rates, all the docks being in the hands of private interests. As soon as the new municipal docks are completed, the control of rates will be reduced to a system.

In Portland, as in San Francisco, pilotage is compulsory, and is an important item of the vessel's expense while in port. The wharves being all private property, there is no public control of rates, and the

charges of various kinds are determined largely by the individual wharf owners, although, of course, there is general conformity to an agreed schedule. The new Dock Commission, by the erection of its own wharves, will naturally assume control of future charges.

In Seattle the newly created Port Commission has the power to fix rates, but has exercised that power for the present only as a matter of form to comply with certain requirements of the statute. Prior to the actual completion of the docks which it contemplates building, it will adopt a thoroughly considered rate schedule. Under the present law it has no legal machinery with which to enforce compliance with its rules, but this defect will doubtless be corrected by the next Legislature.

In Tacoma and other Sound ports, practically the same system and schedule of charges prevail as at Seattle, but it may be expected that, in the near future, an effort will be made to revise the whole system through some mutual agreement.

The British Columbian ports being still almost exclusively under railroad control and ownership, as far as port facilities are concerned, the rates are privately controlled, and the charges are absorbed largely by the railroads.

Co-operation among Ports.—The writer's study of this question has led him to believe that important results might follow a duly organized conference among the recognized authorities of the various ports on the Coast with a view to evolving a uniform system of port administration and of port charges.

V.—Plans for the Future.

Let us now take a brief survey of the ports of the Coast from the viewpoint of their plans for the future. In the first part of this paper it was stated that the Coast is putting its house in order for the reception of the trade which will come from the opening of the Panama Canal. It is a safe assumption that nine-tenths of the funds provided in the past three years, or that will be provided in the next three, for the development of the Coast ports, have been secured through the argument that they are necessary for the above purpose. Ambitious plans have been prepared by recognized experts looking to a future a hundred years ahead and involving expenditures which can only be guessed at. To a great extent these plans will slumber indefinitely, because of

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TABLE 6.—COMPARATIVE COSTS TO SHIP AND CARGO IN PACIFIC COAST PORTS.

	San Diego.		Los Angeles.		San Francisco.		Portland.		Puget Sound.		Vancouver, B. C.	
	Rate	Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	Rate	Amt.
Pilotage (in)	$3 per ft. draft ..... 3 cents per ton	$270	$1 per ft. draft ..... 1 cent per ton	$90	$3 per ft. draft ..... 3 cents per ton	$270.00	$4.50 per ft. draft ..... 3 cents per ton	$315	$125 to $175.....	$150	$1 per ft. draft ..... 1 cent per ton	$90
Water, survey and misc. (estimated)	.....	50	.....	50	.....	50.00	.....	50	.....	50	.....	50
Dockage, per day	$2 plus 3¢ cent for each ton over 200. ..	285	For ship over 1 200 tons... $14.75	177	Discharging: $4 plus 3¢ for each ton over 200. Loading: ½ of above...	427.50	None	0	None	0	None	0
Stevedoring, per ton	4 cents	9 600	40 cents.....	9 600	40 cents	9 600.00	40 cents.....	9 600	40 cents.....	9 600	40 cents.....	9 600
Handling, per ton	Included in "wharfage."	.....	41.8 cents (av.)..	10 032	25 cents.	6 000.00	Included in "wharfage."	.....	Included in "wharfage".	.....	Included in "wharfage"	.....
Dock rental, per linear ft. per month...	0	.....	0	.....	45 cents.	225.00	.....	.....	0	.....	0	.....
Wharfage, per linear ft. per month.....	50 cents	12 000	6½ cents (av....	1 560	5 cents (24 hrs.)..	1 200.00	50 cents.....	12 000	50 cents (120 hrs.)	12 000	50 cents.....	12 000
Wharf storage, per linear ft. per month...	.....	.....	.....	.....	5 cents (48 hrs.)..	.....	.....	.....	25 cents per ton per mo. or frac. mo.	0	.....	.....
Pilotage (out)	Same as "in" ..	270	Same as "in"...	90	Same as "in"....	270.00	Same as "in".	315	Included in charge for "in" pilotage.....	0	Same as "in"	90
.....	.....	$22 475	.....	$21 599	.....	$18 042.50	.....	$22 280	.....	$21 800	.....	$21 890
Cost paid by ship.....	.....	$875	.....	$407	.....	$1 242 00	.....	$ 680	.....	$200	.....	$290
Cost paid by cargo.....	.....	21 600	.....	21 192	.....	16 800 50	.....	21 600	.....	21 600	.....	21 600

This table is the result of a comparative study of the rates in the different ports with a view of determining the relative tax on commerce in each. In order to arrive at a condensed approximation of the cost of terminal charges and a comparison of the principal Pacific Coast ports, a purely hypothetic case has been assumed of a 6 000 net ton register ship, 36-ft. draft, discharging and loading cargoes of 12 000 tons of miscellaneous freight, the ship remaining in port 12 days. It is admitted that the assumption is not typical of ordinary occurrences, but it is apparent that, because of the complex and diversified tariffs, as well as lack of uniformity in freight staples, it is well-nigh impossible to obtain averages except through an assumption that is somewhat strained. The cost of stevedoring has been taken at the uniform rate of 40 cents per short ton at all the ports. This is not strictly true, but the labor unions may be relied on to equalize the cost by dictating the labor wage and the length of a day's work. Small miscellaneous items like water, etc., are insignificant, and have been lumped in the table at $50 per voyage. The charge for handling is a variable sum. The attempt has been made to arrive at an equitable average for each port. The dock rental, in San Francisco, for lines having regular assignment, is 45 cents per month per linear foot of dock rental. A 500-ft. space will amount to $225 per month. A monthly sailing for the case docketer will make the rental $225 per voyage.

their very excess over any actual requirement; but nevertheless there will be an immense stride forward in definite development. How far it will extend in the near future will depend on the progress made before the tide of popular enthusiasm begins to ebb, as it may be expected to do within a few years after the opening of the Canal, and the shrewd promoters of port development and no less shrewd port engineering experts will doubtless make the most of the present opportunity and ride on the crest of the flood wave to a point which the normal depth of water would not permit them to reach.

San Francisco Bay.—San Francisco has a fund of $10 000 000 for port improvement, embracing sea-wall and dock construction and the acquisition of 64 blocks of submerged tide lands. A considerable portion of this sum will probably have to be used in replacing defective structures, but it is already decided to build twelve additional wharves of dimensions varying from 600 to 850 ft. in length by 140 to 200 ft. in width. Six additional ferry slips are to be provided, and the two sections of the belt railway are to be connected across Market Street. The approaching Exposition will doubtless stimulate to still greater exertion, particularly if San Francisco secures home rule in the management of her port.

Across the Bay, the City of Oakland has voted $2 503 000 in bonds, and is actively at work in developing her west water-front and her important interior harbor. Her west front development, or the Key Route Basin, is a promising scheme, involving the filling in of 400 acres of land out to the bulkhead line between the Oakland and Key Route Moles, and wharfing out beyond this, still between the moles, to the pier-head line, in such a way as to form an interior basin with numerous slips and quays.

The plans for the interior harbor are scarcely less ambitious, while Alameda, Berkeley, and Richmond have their own projects of development. These have not yet taken form, however, in actually adopted plans or funds for carrying them out.

Los Angeles.—While Los Angeles has fought and is still fighting an heroic battle for the right to build and control a port of her own, she has as yet made only a small expenditure on her own account. She has, however, voted $3 000 000 in bonds and has committed herself to an expenditure of $7 000 000 more. She has had various studies made for the future development of the port, and E. P. Goodrich, M. Am.

Soc. C. E., has presented plans both for the outer and inner harbor which, if correctly reported by the press, are better fitted for a population like that of New York City than anything which even so thriving a city as Los Angeles has any right to expect in the near future. Fortunately, those dazzling schemes are harmless, and doubtless do good by broadening the public vision to a larger horizon.

The Goodrich studies provide an outer harbor in front of Terminal Island protected by a 20 000-ft. extension of the breakwater, and furnishing ten long piers, 500 ft. wide, with intervening slips 400 ft. wide. The inner harbor studies are more immediately practical, though scarcely less ambitious; but, as they involve several alternative plans and have received no official action, it is scarcely worth while to discuss them here. It is enough to add that the actual realization of this plan for 82 miles of developed water-frontage will cost from $75 000 000 to $100 000 000.

According to the latest report of the U. S. Chief of Engineers, the City "is preparing to undertake immediately the improvement of 79 acres on the westerly side of the entrance channel * * * rendering available about 9 100 feet of deep-water frontage." This is in the outer harbor.

San Diego.—In San Diego no comprehensive scheme of development has yet been prepared, but the City has voted the sum of $1 000 000 for new docks and for sea-wall construction. The length of berthing space to be provided with this fund is 1 600 ft., and the length of the sea wall is to be 2 500 ft. This will make possible the reclamation of 50 acres of tide flats. No definite steps have as yet been adopted for a terminal belt railway. It is stated that the City intends to follow up its present plans with a $3 000 000 bond issue. Perhaps most important, as bearing on the future of the port, is the project, said to be in contemplation, of building a railroad directly east to Yuma.

Portland.—Portland, like San Francisco, is pursuing the sane course of "procuring her cloth before cutting out her garments." In 1910 a Commission of Public Docks was created by City Charter amendment, and the sum of $2 500 000 was voted for the construction of such docks as might be needed in the near future. This Commission employed a board of experts from New York City, consisting of E. P. Goodrich, M. Am. Soc. C. E.; W. J. Barney, Jun. Am. Soc. C. E., and Charles W. Staniford, M. Am. Soc. C. E. A complete de-

tailed plan has been prepared by the Board and has just been made public. Those features of the plan which are proposed for immediate adoption consist of two combination docks, sheds, and warehouses, one on each side of the river, a coal dock, a motor-boat landing, fireboat station, etc.

The West Side improvement will consist of a quay dock, 1 075 ft. long and 100 ft. wide, in the central section of the harbor below the city bridges. For two-thirds of its length it will be a single-level structure, and the rest two-level, the upper being at Elevation 32 and the lower at 18 ft. above low water. The dock will be provided with adjustable slips, escalators, and elevators, and will have space for storing 10 000 tons of freight. A six-story reinforced concrete warehouse provided with full equipment for handling goods will be erected in the rear of the dock. Its capacity will be about 40 000 tons. Three additional warehouses are planned as soon as funds are available. Dock autos will be the main reliance for handling freight.

The East Side improvement, conveniently located for the transaction of business, consists of a two-story quay structure, 500 ft. long by 100 ft. wide, capable of storing 6 000 tons on each level. A one-story reinforced concrete warehouse will be provided with a foundation for five additional stories. All structures in both improvements are to be of steel and concrete, and fire-proof throughout.

In addition to these improvements for immediate adoption, the plan of the Consulting Board provides for future development on the two sides of the river, 7 000 ft. of quay construction, pier and slip development affording 21 200 ft. of berthing space with piers 300 ft. wide, together with the necessary warehouses and other accessories. A large coal-handling plant, freight-assemblage and storage yard with a capacity of 1 500 cars, and a public belt line serving all the docks, are also to be provided. The total development gives more than 6 miles of berthing space, of which only about 1 750 ft. are as yet provided for. As to this initial programme, it is understood that it is acceptable to the Dock Commission, and that steps will be taken immediately to carry it into effect. In connection with the Dock Commission projects, reference may again be made to the joint projects of the Government and the Port of Portland Commission to secure a 30-ft. channel to the sea.

Puget Sound and Grays Harbor.—North of the Columbia River, port-development work will doubtless always be associated with the name of Virgil G. Bogue, M. Am. Soc. C. E. Pursuant to a charter amendment of the City of Seattle, adopted March 8th, 1910, a Civic Plans Commission was created for the purpose of forming a plan for the future development of that city. This Commission employed Mr. Bogue as its Engineer, and his very elaborate report, submitted August 24th, 1911, includes a complete scheme of development of Seattle's harbor. Following immediately on the submission of this report came the creation of the Port Commission of Seattle, specifically charged with the duty, among others, of preparing a "comprehensive scheme" of port development. While Mr. Bogue's plans as a whole have not been adopted by a vote of the people, their essential harbor features will be given close study in the preparation of any plans that the Port Commission may adopt. Mr. Bogue, after the completion of his Seattle work, made a similar study for Tacoma and also for Grays Harbor, and he is now engaged on a plan for Prince Rupert. As already stated, whatever action may be taken as to the immediate adoption of his work in these ports, it is certain to have a profound influence on their future development.

The Seattle plan embraces elaborate suggestions for every part of the harbor, with especial emphasis laid on the Harbor Island and Smith's Cove sections. The mosquito fleet and industrial features of the inner harbor are likewise fully elaborated. No effort was made to form an estimate of costs, for the plan is one for long-continued development; but it certainly represents an outlay of not less than $25 000 000.

The Port Commission has definitely adopted an initial programme involving an expenditure of $6 000 000. This includes a slip with two wharves on the east side of East Waterway, giving berthing space for four large ships; a solid filled pier on the central water-front, with an 1 100-ft. berthing space of deep-sea depth, and a floating landing stage for small boats with 800 ft. of landing space, and also a large fruit cold-storage plant; a lumber, coal, grain, and general transhipment dock at Smith's Cove; a small dock for general use on Salmon Bay; and a new ferry across Lake Washington. The Port Commission is taking initial steps looking to the establishment of a terminal railway system for the port.

In addition to the Commission's original programme, there is being considered, at the earnest request of certain business interests, a very ambitious project for Harbor Island, patterned after the Bush Terminals of New York. The pier development proposed comprises six new piers with an aggregate berthing space of 16 800 lin. ft., or enough for the simultaneous berthing of 42 ships of an average length of 400 ft. Back of the piers is to be an extensive industrial development. The whole project is to be a co-operative one between the Port District and a private terminal company.

It is conservative to state that expenditures of all classes—Federal, State, County, City, Port District, and private—actually authorized or in definite contemplation, for the development of Seattle Harbor, aggregate approximately $20 000 000.

The study made by Mr. Bogue of the possibilities of Tacoma Harbor may best be told in his own words, as summarized in his report on Grays Harbor:

"At Tacoma, a great movement is on foot to keep the city well to the fore in all port extension matters. The plans contemplate a canal system with an aggregate length of 6 miles, to say nothing of spurs and laterals which may be built in future years in connection with large industrial enterprises. The cost of this canal system is estimated at about $4 000 000. It will develop a vast area of tideflats and present unexcelled opportunities for industrial development with convenient access from both sea and land. Beside the canal system a number of waterways are to be built and various slips and piers which will provide the best accommodations for vessels engaged in ordinary commerce. When these improvements shall have been made, the wharf frontage of Tacoma between Point Defiance and Brown's Point will aggregate 42.6 miles, all of which is well protected and easy of ingress or egress."

Olympia, Everett, and Bellingham are not as yet taking positive action in the development of their ports, but they will probably do so in the near future.

Grays Harbor.—The Port Commission of Grays Harbor, through Mr. Bogue, has prepared an elaborate scheme of development, but the report does not furnish an estimate of its probable cost, and no provision is yet made for carrying it into effect.

Vancouver, B. C.—As already stated, no comprehensive scheme of development of this port has been adopted. Special studies have been

and are being made of different parts of the harbor, and there are rumors of gigantic development plans by private interests. The Great Northern Railway has a fine new pier under construction, and other private improvements are in progress; but there is as yet no general plan, no distinctive scheme of port administration, and no provision of public funds.

The conformation of the harbor and the handicap of tidal fluctuation and currents have suggested an improvement which, though costly, would seem to possess great merit. That is to place a lock and dam across the throat of the harbor at the second Narrows, thus holding the upper part of the Inlet at high tide and making it a fixed-level fresh-water harbor. This would restrict the tidal area to one-third its present extent and would reduce tidal currents in the lower harbor so much as greatly to improve conditions there. It would seem on the face of it that this would be an improvement of great importance to the future of the port.

New Westminster.—The fresh-water port of New Westminster is taking active steps in harbor development. It has recently expended $15 000 on the study of a plan and its presentation, and is about to put into effect the one prepared for it by A. O. Powell, M. Am. Soc. C. E. It comprises wide marginal streets, routes for railways, the extension of the city quay, and a long waterway in a slough parallel to the river, from which the river is to be excluded at the upper end. A succession of parallel slips, oblique to the axis of the waterway, will develop a berthing space of 7½ miles.

Victoria.—Even that sometime staid and delightful provincial town of Victoria, more distinctly British than any other port of the Pacific, has become infected with the Panama *bacillus,* and is cogitating what it may do to be prepared for the changes which it has been told are about to ensue. No definite plans have as yet been evolved, but among the tentative suggestions are an outer harbor to be protected by a breakwater in front of the present entrance to the inner harbor, and also a development in Oak Bay on the opposite side of the town fronting to the eastward.

Prince Rupert.—Studies for the Port of Prince Rupert are now in progress under the direction of Mr. Bogue, but are not yet far enough along for incorporation in this paper. A suggestion that has been made of possible future development in this port, however, may be

properly mentioned. Prince Rupert City is on an elliptically shaped island about 6 miles long north to south by 4 miles east and west. The "lakes" which separate the island from the mainland in the rear have an area north of the railroad crossing of about 40 sq. miles. It is not unlikely that the north and south passages will be dammed off and provided with locks, thus converting the lakes into a tideless, fresh-water harbor just at the level of high tide. While this development is still some distance off, it has such great advantages, in view of the extreme tidal fluctuation in Prince Rupert Harbor, that it will probably be realized eventually.

Outlay on Pacific Coast Ports.—From the statistics of obligations actually assumed to date, and by the aid of the most reliable forecasts obtainable of port planning for the near future, it will be a conservative estimate to place the cost of port development along the Coast during the next five years at $50 000 000.

VI.—Influence of the Panama Canal.

The writer will close with a brief statement of the influence ot the Panama Canal on the commerce of the Pacific Coast. The aggregate effect will be a summation of the following:

(1) Effect on consumption and production among the existing population;

(2) Effect on consumption and production through increase of population—chiefly immigration;

(3) Effect on rail commerce between the Pacific and Atlantic Coasts;

(4) Effect on industrial development;

(5) Effect on Oriental traffic to the east coast of America now transhipped to rail at Pacific Coast Ports;

(6) Adverse effect of Canal tolls.

Consumption and Production.—Under the first heading, it may be assumed that there will be some increase of consumption due to a reduction of prices resulting from lower freight rates *via* the Canal. For example, anthracite coal may possibly find considerable use along the northern coast, where now it finds very little, if any at all. As to production, the effect ought to be extensive and immediate on lumber products, for the Canal will open to Coast lumber the market of both shores of the Atlantic, and the production can be instantly increased

to any desired extent by putting existing mills now closed into operation or speeding up others to their full capacity. In the matter of fruit and cereal products, no radical effect on production should be expected as an immediate result of opening the Canal. The conditions are not the same as with lumber, and any material increase of production must await an increase of population and general development.

Immigration.—This consideration brings up the second heading specified—the influence of the Canal on the increase of population on the Pacific Coast. It is an influence which, in the long run, may prove the most important of all; for it is not so much additional port facilities that are needed to develop a country as increased numbers of those who do the work of development. Many other factors, of course, contribute; but in a new country increase of population is the main thing, and this must come in large part through accessions from the outside. There is a general consensus of opinion that the opening of the Canal will have more immediate effect on immigration than on any other class of traffic. In this fact lies a great hope and a great peril for the Coast. What kind of immigrants will be brought here? What reception will they meet? Will they prove a blessing or a curse to our communities? The writer will consider these matters no further than to quote a paragraph from a bulletin recently published by the Seattle Port Commission:

"In this connection the commission may be justified in digressing so far as to observe that the real preparation for this influx of immigration rests with the people of the State rather than with the Port Commission or the Government. If intelligent and organized effort is made through agents in Europe to secure a good quality of immigrants, and if similar effort is made here to start them in their new career without too much hindrance and discouragement and without being victimized by those whose only desire is to fleece them of their meager belongings and leave them stranded in a country where the problems of successful agriculture are so different from what they are accustomed to, this immigration may be of inestimable value to the State and indirectly to its metropolis and chief port. But if this matter is allowed to drift along without organization or sympathetic effort in aid of the new arrival, the result may be to crowd this port with an indigent population and to justify the effort which is said to be already under way here to resist the whole immigration movement."

Diversion of Rail Traffic.—In reference to the third consideration, a degree of uncertainty exists which completely nonplusses the ablest

traffic authorities of the country. The rail traffic between the Pacific Coast and points far enough east to be affected materially by the Canal is smaller than is popularly supposed. A high railroad authority has estimated that not more than 600 000 tons of traffic goes to the Northwest Coast from east of "Missouri River common points", and that only about 20% of this originates east of Buffalo or Pittsburgh where the influence of the Canal would be effective. On traffic from the Great Lakes and from the Mississippi, Ohio, and Missouri Basins, it is not likely that the Canal will have much effect. The railroads will hold it, except for the possibility that it may cease to originate in those sections. All but about 7½% of this traffic, it has been estimated, might originate on the Atlantic and Gulf Coasts, where it would fall directly within the Canal influence. Is it not likely that the ultimate result of the Canal will be to transfer to the Atlantic Coast much of the Pacific Coast traffic which now originates in the central sections, thus cutting off from both the railroads and the producer of the Central West an important business? And is it not a perception of this probable result which accounts for the lukewarmness, if not active opposition, of certain sections of the country to free tolls for coastwise trade through the Canal? In any event, it seems to be reasonably certain that the Canal (particularly with free tolls) will transfer from rail to ship a large part of the traffic which now crosses the country to the westward by rail. This will operate to increase the business of ports on both coasts and of the railroads from the coast inland for uncertain distances.

As to east-bound traffic originating on the Coast, lumber shipments will increase very largely, but rather through the opening of new markets than the diversion of existing rail shipments. The present eastern market for Coast lumber is mainly in the tramontane region eastward as far as the Mississippi. The market farther east cannot stand the heavy freight charge, except for shingles and the finer grades of lumber. The market of the Central West will continue to get its supply by rail, and only consignments farther east are likely to be diverted to the water route.

With fruit products of all kinds along the Coast, there is every reason to believe that the Canal will effect a marked change in the method of shipment. Refrigerator ships will carry this product to both shores of the Atlantic in better condition and at cheaper rates

than can be done under present conditions. It is estimated that within 10 years the fruit shipments from Central Washington and Oregon will approach 100 000 carloads, while the shipment of all classes of fruit from California will exceed that figure. The problem of handling those prodigious quantities by rail in the short periods and over the long distances necessary may prove an impossible one; and the water route will come in as a great relief in addition to its normal advantages. It seems to be one of the certainties of the near future that cold-storage plants of immense capacity will be built in all the principal ports in order that the fruit crop can be stored in part, awaiting shipment.

In regard to wheat and other grain, and fish products, the Canal will not make much change, because these products already go largely by water; but considerable change in the method of water shipment is likely to result. Wheat, for example, is now shipped mainly in sacks in order to avoid the alleged danger of heating if shipped in bulk on the long voyages *via* the Horn, Good Hope, or Suez. The shorter voyage *via* Panama will obviate this difficulty, and we may expect to see elevators in greater numbers provided in our ports, as they are on the Great Lakes, at Montreal, and in other Eastern ports.

Industrial Development.—It is a much disputed question as to what the effect the Canal will have in promoting industrial development on the Coast, particularly manufactures. Some hold that the lower freight cost will bring raw materials here at rates which will make manufacturing practicable where it is not at present. On the other hand, it is argued that the lower rates will bring manufactured products from the East more cheaply than at present, and that this will offset the tendency just noted. Where the balance of these conflicting forces will fall is wholly problematical, but it is certain, from the uniform experience of the older sections of the country, that increasing growth in population and wealth of itself will bring increased development of manufactures. It will become a part of the development of every port, and will operate to increase its business.

Oriental Traffic.—As to Oriental traffic transhipped to or from Atlantic or Gulf points, it is clear that the effect of the Canal must be to draw it away from Pacific Coast ports. It will not draw it away altogether, because, with certain commodities, like silk, of high value in proportion to freight charge, the interest on purchase cost may amount to more, through the longer time required to receive the goods *via* the

Canal, than the saving in freight. Most of the manufacturers consulted are agreed that silk shipments will continue to go by rail. With other articles, however, such as hemp bound East and cotton bound West, it would seem that, if through lines are established direct to the Orient, *via* Panama, they must absorb this class of traffic.

Canal Tolls.—It is a matter of profound satisfaction to be able to state in this discussion of the ports of the Pacific Coast that a question vital to their welfare and one but yesterday of doubtful issue has been settled and settled right. Our coastwise trade will pass through the Canal without paying any toll. The great principle of the free public highway, and of the free use of public improvements of rivers and harbors, has triumphed over the narrow vision and selfish interest that threatened to defeat it. It is one of the greatest victories of an enlightened national citizenship, which illumines the record of this generation.

The victory, however, great as it is, is not as great as it should be. The Canal should be free to all commerce. The highway toll system on this great thoroughfare should be abolished for all nations and forever. The same system on the public roads became its own executioner, because it defeated its own end. It prevented the use for which the road was built, and while it was an improvement on no road at all, it never performed its full service. So likewise, tolls on commerce through the Panama Canal will restrict the rightful use of that great work. This is a demonstrable certainty. Take a table of distances and look up, for example, the routes from New York to Valparaiso *via* Panama and Magellan. The Panama route is 3 500 nautical miles shorter. Yet it is a fact that the delay in passing the Canal plus the toll cost at a dollar a ton commuted into cost of running the vessel will make it cheaper for large fast vessels to take the longer route. To vessels between Valparaiso and European ports, the disparity against the Canal route will be still greater, and, of course, the same principle applies in varying degrees to all routes. Tolls will inevitably restrict the use of the Canal, and thus deprive the United States and the rest of the world of its full benefit.

It is one of the most powerful illusions that public thought is subject to, that a work like this, in order to justify itself, must be made to pay its own cost in direct revenue. It is easy to overlook the fact that every dollar collected in this way has to be paid by the people in

some other form. At the very best, it is taking out of one pocket to put into another, and meanwhile the utility of the work is being curtailed by restriction of use. Happily the true principle has been grasped by our legislators and applied to the Canal, as far as our own trade is concerned, and that is the utmost which it was in their present power to do. It is only just that foreign nations which have borne no portion of the cost of the Canal should pay something for its use. No one can question the correctness of that position, but this end should be attained by payment for this use, without imposing a toll, by the discovery of some method which will not restrict the use of the Canal. Is such an achievement possible? The writer believes that it is.

It can come, of course, only through international agreement; but it is conceivably practical to form a convention by which each nation shall pay into the Treasury of the United States a proportion of the rightful charge based on the actual use of the Canal by its own subjects, or under its own flag, or on the tonnage to and from its own ports. The charge would be computed on cost of maintenance and operation and such an additional sum annually as would retire the indebtedness within a determinate period. The question is surely worthy of highest consideration, and few subjects more vital to the interests of the world at large could engage the attention of the Third Hague Conference in 1915 than this of mutual co-operation to make the greatest public work of the ages free to all users. Let the police jurisdiction, direct control and operation, and questions of use in time of war be left to the United States; but otherwise let it be free. As the writer has elsewhere said, a "way must be found for making Panama and Suez as free to the world's commerce as if Nature herself had excavated their channels—as free as are the Straits of Gibraltar and Magellan."

DISCUSSION

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—On page 180 the authors speak in glowing terms of the future possibilities of the Lake Washington Ship Canal scheme, now well under way. As a matter of historical interest, it may be stated that, in October, 1871, the late Gen. B. S. Alexander, accompanied by Lieut. (now Col.) Thomas H. Handbury, M. Am. Soc. C. E. (both of the Corps of Engineers, U. S. A.), and the writer, as Assistant Engineer, went to Puget Sound with the view of making extensive "current observations" for the purpose of determining the practicability of harbor defense by torpedoes. While on this mission, Mr. Briar Brown and Dr. Whitworth, two estimable citizens of Seattle, took the party to the proposed ship canal site, and explained the scheme. Orders were received to make the survey, and called for plans, estimates of quantities, and probable cost of making a ship canal,

"with the view of ascertaining the adaptability of this lake for a naval depot, and the proper route for a ship canal to connect it with Admiralty Inlet, and the cost of such canal.*"

The field work was done by Lieut. Handbury and the writer. The report of the results of this survey will not be found in the River and Harbor Index, because the expenses came out of "Surveys for Military Defense"; consequently, it will appear only among the Senate Executive Documents for the fiscal year ending June 30th, 1872.

At the time this survey was made, it was claimed that the main object of the fresh-water depot was to enable the naval fleet to go up into Lake Washington, and, while taking on stores and supplies, all barnacles, sea-grasses, etc., befouling their bottoms would naturally drop off, leaving them clean when they proceeded to sea. In ordinary cases, of course, ships would have to go into dry dock to have their bottoms cleaned. For large vessels, this would be a matter of $2 000 per day in dock expenses, hence the great saving to be naturally expected. The writer has no doubt whatever of the great future possibilities of the Lake Washington Ship Canal, and will watch its growth in usefulness with interest.

Speaking of the Sound waters in general, the writer was greatly impressed by the bold shores everywhere. In fact, a 3 000-ton ship can go up to the shore, make fast to the trees, and lay there as long as necessary. This same feature was also noted in Lake Washington, and the writer is firmly of the opinion that it is due to vertical stratification; thus, whenever one stratum drops off, it always leaves a new vertical face standing. This natural feature makes trouble for

*Report, Chief of Engineers, U. S. A., 1872, p. 26.

Mr. Le Conte.

the commercial wharves along the shore, but, nevertheless, it has its advantages in some cases.

The removal of Blossom Rock, in San Francisco Harbor, was a most interesting piece of work, and the fact that it ended disastrously was due, as usual, to unexpected contingencies. The writer had the pleasure of visiting the work several times during its progress, and just after the blast, in April, 1870, was told by the late Col. A. W. von Schmidt, the contractor, that he was compelled to set it off some time before he really intended, because of the exceedingly shaky condition of the rocky shell overhead, which had to be propped up with timbers. The tremor due to the working of the hoisting engine at the main shaft would start up leaks everywhere, and he was dreadfully afraid of a general collapse; hence the blast was premature. The subsequent quantity of dredging and scraping necessary to get the required depth of 24 ft. at low water ate up all the profits on the contract.

The draft of vessels entering the harbor gradually increased to 28 ft., and in November, 1902, a second contract was let for increasing the depth over the rock to 30 ft. at low water. The extra 6 ft. to be removed being largely loose broken stone, the contractor resorted to surface blasting, and excavating with a clam-shell dredge and a 10-ton bucket of the grapple type with long lever arms. The results were highly satisfactory.

The same system and dredge were also used in the removal of Shag Rock No. 2, and the results were equally satisfactory. The whole secret of success in surface blasts lies in the use of small charges, exclusively. The writer had local charge of the removal of Rincon Rock, in San Francisco Harbor, and the contractor engineer, Mr. Albert Boschke, made the great mistake of using 40-lb. charges for surface blasting before dredging the broken rock. Such large blasts simply expend their force in making a fine fountain, and the mechanical effect on the bottom, where it is needed, is practically nothing.

The quantity of powder required for each blast is that which will just begin to make a fountain. In most cases this is about 6 lb. of, say, 90% nitro-gelatine. These small blasts, judiciously distributed and followed immediately by clam-shell dredging, are by far the cheapest and most effective, where applicable.

A few remarks about the great jetty at the mouth of the Columbia River may not be out of place. The prodigious movement of sand mentioned in the early reports on this work proved to be a myth, and later experience showed that the sand simply traveled in an orbit of limited diameters. When the Point Adams Jetty was well extended, the sand movement practically disappeared, because it was "bottled-up." The depth of 31 ft. was obtained over the bar, and everybody rejoiced. Peacock Spit, on the north side, next to Cape Disappoint-

ment, was practically the natural north jetty. Unfortunately, Peacock Spit was cut away by subsequent heavy storms and strong currents, and as a result the bar shoaled up to its original depth of 21 ft. at low water. It is very evident that the only thing to do is to build another jetty from Cape Disappointment out along the crest of Peacock Spit, and thus maintain the spit and hold the tidal currents up to their work along and against the south jetty. The writer is of the opinion that a 31-ft. channel cannot be maintained unless Peacock Spit is fixed on the north side. Mr. Le Conte.

Dredging has grown to be a most important adjunct to every type of harbor work. The great advances made in machinery have cheapened the cost of dredging to such an extent that all schemes for proposed harbor work are now materially affected thereby. It may be stated that the day for extending jetties out into deep water in order to maintain the depth on the bar is gone forever. The annual interest on the cost of deep-water extensions will pay for the necessary dredging five times over.

The improvement of tidal flats for commercial purposes is looming up everywhere. When the fact is considered that the new property, after full improvement, can be readily sold for $30 000 per acre, one can see how easy it is to pay for proposed schemes.

Everywhere in Europe it is customary for the government to condemn and purchase all property contiguous to a proposed harbor work, and, after improvements are completed, sell the property to the highest bidder. As a result, it not only gets all its money back, but in addition a handsome profit. The writer fails to see why the same cannot be done in the United States.

The work for bar dredges is growing more important every day. Where the entrance is largely under good shelter, the work can be done for 4 or 5 cents per cu. yd.; but where the bar is exposed to heavy deep-sea swells a large part of the time, the working period is cut down fully one-half, and the cost naturally goes up to 8 and 10 cents per cu. yd. These results are so flattering that harbors of any importance cannot afford to get along without a good bar dredge for general use, as it will pay back its first costs in one year's operation.

E. P. Goodrich, M. Am. Soc. C. E.—Having been a student of port problems for some years, and being intimately connected with some of the work described in this paper, the speaker has studied it with considerable attention. Concerning several points, his interest has been specially aroused. The general problem of the Panama Canal and its effect on the commerce of the world is of great interest, and, together with the various physical problems discussed by the authors, has been of professional interest to the speaker. On these matters, however, he does not care to make any comments. Mr. Goodrich.

Concerning several non-professional points, the speaker's interest

Mr. Goodrich.

has been similarly aroused. Among these is the fact that, in a professional paper before the American Society of Civil Engineers, nothing but an obviously exaggerated newspaper report has been used in describing the port conditions at Los Angeles, Cal. The speaker may lay himself open to criticism for possessing unprofessional curiosity, but he must confess that had it been possible, he would have liked to await the authors' final discussion in order to ascertain whether or not they would correct the descriptions of the projected Los Angeles work, by making use of modified but obviously more accurate subsequent newspaper articles, to the files of which they evidently have access. The speaker is in entire accord with the opinion expressed in the paper, that it is possible that the popular enthusiasm which is now working such wonders on the Pacific slope may go too far, and that the pendulum may eventually start on a reverse swing. Thus, both in Portland, Ore., and in Los Angeles, Cal., he has laid himself open to the possibility of becoming unpopular by sounding a strong note of conservatism. It may not be out of place to state that the newspapers in both cities reported such fact, which might have been discovered by the authors had they read the local papers carefully. Because of the juxtaposition of the speaker's name, coupled with certain comments about the Los Angeles work, with the authors' reference to "shrewd promoters of port development and no less shrewd port engineering experts," who "will doubtless make the most of the present opportunity and ride on the crest of the flood wave to a point which the normal depth of water would not permit them to reach," he is afraid that casual readers will obtain an erroneous conception of the Los Angeles work and of his connection with it. Even the qualifications inserted by the authors are considered inadequate to meet the exactions of truth, and, therefore, the following facts are presented to the Society in an effort to secure justice for a city which the authors themselves have characterized as "incontestably the center of activity of Southern California" and destined to "become a great port, not because Nature made it so, but because her own virile people have said so."

The facts (which might have been ascertained by correspondence or by perusal of public prints) are as follows: The City of Los Angeles owns about 200 acres in the outer harbor, comprised in two tracts of about 150 and 50 acres, respectively. The outer harbor works now contemplate the immediate improvement of only one of these tracts, work on the 50-acre tract being practically under contract to the extent of bulkheading and filling, while plans for the 150-acre tract are in process of evolution, the detailed design being made subject to the determination of certain physical conditions with regard to currents, the occurrence of rock in the foundations, and the securing of certain real estate to provide rights of way for means of access.

Mr. Goodrich.

In a professional paper* published nearly a year ago by Amos A. Fries, M. Am. Soc. C. E., Captain, Corps of Engineers, U. S. A., a suggestion was made with regard to the possibility of further improvement of the outer harbor, proposing a tremendous breakwater and certain long piers similar to those described by the authors and ascribed to the speaker:

"* * * due to the fact that the ocean bottom becomes flatter as you go east from Point Fermin, the breakwater can be extended to inclose any amount of anchorage that may be desired. Indeed, a 25 000-foot extension, should that much ever be required, could be made on almost the same line as the outer arm of the present breakwater, and while keeping in depths averaging barely 48 feet would inclose 10 square miles of water, half of which would average more than 36 feet.

* * * * * *

"Not only can the breakwater be greatly extended, but if fifty or one hundred years hence a long extension becomes necessary, the harbor frontage itself can be increased at least 17 miles by the construction of nine slips between Deadmans Island and the entrance to the Long Beach Harbor. The slips beginning at the present 18-foot curve could be made in lengths varying from 6 500 to 2 100 feet, with a tongue of land 1 000 feet in width between each two slips.

"It would seem advisable, whenever any considerable extension of a breakwater is made, to leave a gap 2 000 feet in width between the present breakwater and the beginning of the extension."

While the authors state that great reliance is being placed on the inner harbor in the development of port facilities, they also say that:

"A great saving fact in the enormous labor of building the Harbor of Los Angeles is its close relation to the industrial development of the city. The lands into which the waterways are being dredged are admirably adapted for factory locations, and the material of excavation is being used in making the necessary fills. The whole development goes hand in hand in a way to produce the best results."

It is the speaker's firm opinion that the inner harbor will be developed almost exclusively for industrial plants and the outer harbor for deep-sea tonnage. At the wharves of the inner harbor, vessels with full cargoes for special manufacturing plants will berth. In order to make the best use of certain dredging which has already been completed, the speaker recommended, and it is understood that the contracts have now been let for, certain temporary wharves located on one of the arms of the inner harbor as now laid out. Only a portion of this wharf will be shedded, and this whole improvement will be carefully studied in an effort to determine what is likely to be the future of that portion of the port. It is further believed that the authors should have put their qualification with regard to the profitableness of reclaiming tide flats, noted under the head of "Dredging,"

* "Los Angeles Harbor," *Professional Memoirs*, Corps of Engineers, U. S. A., and Engineer Department at Large, Vol. IV, No. 13 (Jan.–Feb., 1912), p. 1.

Mr. Goodrich.

in closer proximity to the point they endeavor to make of the great cost of doing the harbor work in Los Angeles. They say:

"Now that the reclamation of tide flats is becoming so profitable an enterprise, dredging will be resorted to more than ever, the operation serving the double purpose of excavating slips and channels and filling the abutting lands."

Increased real estate values will be much greater in Los Angeles in comparison than in many other locations, so that moneys expended will produce larger returns for self-supporting enterprises, such as harbor propositions have generally shown themselves to be, and the criticism of the costly nature of the work at Los Angeles Harbor submitted by the authors becomes largely nullified. In reference to this, it may be well to call further attention to what the authors say with regard to San Francisco Bay:

"The shoaling of San Francisco Bay is one of those great natural blessings which the unthinking are so accustomed to look on as a curse. One-tenth of its natural area, with deep connecting channels, would serve every possible need of commerce, while the other nine-tenths would be of immeasurably greater benefit reclaimed and turned to industrial or agricultural use. Every cubic yard of earth washed down from the rugged slopes of the mountains is worth a thousand times more in those low areas, where it is turned to efficient use in the service of Man."

As to the inner harbor at Los Angeles, the speaker has recommended certain alterations in the harbor lines, a portion of which proposed changes has been presented to the War Department for consideration, and it is hoped will receive its approval. Again, with regard to the inner harbor, such changes in harbor lines are in close accord with the recommendations contained in the paper by Capt. Fries, who writes as follows:

"Between Deadmans Island and the turning basin in the inner harbor there are 18 000 feet of bulkhead lines and 500 acres of land, most of which is now reclaimed. The frontage there may be increased a few thousand feet by slips. Above the turning basin, in the east and west basins combined, there are 750 acres of reclaimed land and 52 000 feet of frontage exclusive of the Salt Lake Railroad Company's land on the south and east. This frontage may be increased economically by slips to the extent of 13 000 feet.

"The above is along present approved harbor lines, and while the total—including the slips suggested by the writer and the 12 000 feet along the Salt Lake Railroad Company's property—amounts to 132 000 feet, or 25 miles, it does not represent half the frontage that can be developed if the future shall show that more is needed.

"Bounded by the bluffs of San Pedro on the west, the Anaheim Road and the city of Wilmington on the north, the City of Long Beach on the east, and San Pedro Bay on the south, there are some 8 square miles (about 5 000 acres) of swamp, tide, and submerged

lands capable of being practically improved as part of the inner harbor. Before this is all developed it is evident that the present anchorage area will be too small. Mr. Goodrich.

* * * * * *

"* * * the land owned by the Salt Lake Railroad Company between the Long Beach Harbor and the east basin can probably best be developed by slips opening into the east basin and the Cerritos Channel between it and the Long Beach Harbor, giving a frontage of 12½ miles. On Plate V the channels are shown 500 feet wide on the north side of Cerritos Channel, where they are about 1 mile in length, and 300 feet wide on the south side where the lengths are 2 000 feet."

The authors are entirely wrong with regard to their criticism of the possibilities of the inner harbor of Los Angeles becoming silted by material carried in the floods of the Los Angeles and San Gabriel Rivers. The speaker cannot do better than quote again from the report of Capt. Fries:

"The question of maintenance of depths is always a very important one when considering the future of a harbor. In this matter Los Angeles Harbor is exceedingly fortunate. Indeed, it is hard to conceive of an ocean harbor that will cost less to maintain. The two causes of the deterioration of a harbor are silt carried down by rivers flowing into the harbor and sand piled up at the entrance by cross-currents and wave action.

"Ordinarily, the Los Angeles River is the only one whose waters reach the harbor during the rainy season. During the greater part of the year the river goes entirely dry before reaching the sea, due to irrigation and the great quantities of water used in the city of Los Angeles. It is noted, however, that during the winter of 1910-1911 the San Gabriel River, which ordinarily flows into Alamitos Bay, about 10 miles east of Los Angeles Harbor, broke from its regular channel into one known as New River at a point about 20 miles from the harbor and, following the New River, united with the Los Angeles River at a point about 5 miles north of the harbor. This was one of the worst floods known in many years and carried into the Los Angeles Harbor possibly 350 000 cubic yards of material. Efforts are now being made by railroad companies and agricultural interests in the vicinity of the break to make such improvements in the bank of the river as will confine it in the future to its regular channel, emptying into Alamitos Bay.

"The Government has been asked to aid in this as a measure of protection to the harbor, and steps are being taken in that direction. Unquestionably this improvement will be made, but even if the San Gabriel River should regularly flow into the Los Angeles Harbor, the cost for dredging would still be comparatively small, as the records for nearly fifty years show only five serious floods. These occurred in 1867, 1873, 1884, 1891, and 1911. The Los Angeles River itself carries down some material in smaller floods at lesser intervals, but the amount is so small as to be scarcely noticeable, except just where the river first enters the deep water of the harbor."

Mr. Goodrich. Another point described by Capt. Fries, and now being actively pushed by the local authorities, is the widening of the channel between the inner and outer harbors to a minimum of 750 ft. A request for this improvement has been formally filed with the War Department, and it is believed that the Washington authorities look favorably on the suggestion.

In discussing the administrative systems of the ports, the authors criticize the conditions in Los Angeles, where there was originally and is now technically a divided responsibility. The trouble described by them, however, has been entirely overcome by the Mayor, who appointed what is called an "Advisory Harbor Commission," consisting of the two boards meeting jointly, with the Mayor as Chairman. During the meetings of this joint board all matters relating to harbor affairs are talked out, and differences of opinion are eliminated. The Mayor's solution of the small difficulty described by the authors has proven highly advantageous to the community.

With reference to the table of comparative costs to ship and to cargo in Pacific Coast ports, the fact should be pointed out that, with the exception of San Francisco, the table shows Los Angeles to have the lowest total cost. It should be noted further that the handling rate of 41.8 cents has been taken from an average which might be vastly different in an actual case. The dropping of the 0.8 cent would reduce the cost by nearly $200.

The speaker is extremely sorry that the authors were not more accurate in their statements with regard to Los Angeles Harbor, and believes that it throws grave doubt on the accuracy of the paper in other points.

Mr. Haupt. Lewis M. Haupt, M. Am. Soc. C. E. (by letter).—This is a very comprehensive paper, not only on the engineering problems of Pacific Coast harbors, but also on their commercial relations; and, in view of interesting discussions previously published in the *Transactions* of this Society,* the present contribution is one of great value to the Profession as a guide to and aid in the early solution of these intricate questions.

History and experience are the foundations of theory on which the engineer must rely to shape his course in order that he may best serve his Profession and his country by removing these obstacles to international commerce, hence it is that this paper is peculiarly appropriate on the eve of the completion of the Panama Canal. As it

* "On the Straits of Juan de Fuca, Puget Sound; and Government Improvements on the Pacific Coast," by B. W. De Courcy, M. Am. Soc. C. E., Vol. XXV, p. 420.
"Improving the Entrance to a Bar Harbor by a Single Jetty," by T. W. Symons, M. Am. Soc. C. E., Vol. XXXVI, p. 109.
"Description of Coos Bay, Oregon, and the Improvement of Its Entrance by the Government," by William W. Harts, M. Am. Soc. C. E., Vol. XLVI, p. 482.
"Seacoast Harbors in the United States," by C. E. Gillette, M. Am. Soc. C. E., Vol. LIV, Part A. pp. 297, 385.
"Notes on the Bar Harbors at the Entrances to Coos Bay and Umpqua and Siuslaw Rivers, Oregon," by Morton L. Tower, M. Am. Soc. C. E., Vol. LXXI, p. 349.

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covers all the important ports from San Diego to Vancouver, no detailed discussion is possible, nor are the maps appended sufficient for such purpose; the text, however, supplements them in large part by stating local conditions and results, and several extracts therefrom are submitted as pointing to such modifications of practice and policy as to give promise of much greater certainty of securing early results.

As it is not possible to review all the cases cited, that of the typical and important Columbia Bar is taken as an example, because, as the authors very justly state:

"This work * * * is probably the largest and most difficult of its kind ever attempted * * *. The two great obstacles to be overcome * * * are the storms and the teredo * * *. The roughness of the sea precludes the use of barges for dumping rock, thus necessitating a trestle, and the trestle piling is the particular delight of the teredo, which puts it out of commission in from 10 to 20 months. * * * the embankment is progressively shaken down during each winter season."

Specifically, the work is described as follows:

"The jetty was to be built of large size stone on a brush mattress and raised to the level of low tide. Later, the project was changed to raise it to high tide, and four groins were to be built from the north side to arrest scour. Work was begun in 1885 * * *, and was completed ten years later at half the estimate. The depth over the bar had increased, in the meanwhile, from 21 to 31 ft., and the work seemed to have accomplished its purpose perfectly.

* * * * * *

"Several years later there was only the original depth of 21 ft. * * *. A new project was adopted, extending the jetty 3 miles farther, * * * to be supplemented by dredging over the bar. * * * a permanent depth of 30 ft. is sought.

"The jetty itself, as now being built, consists of a bed course of small rock as a substitute for brush mattress, it being found impracticable to use the mattress beyond the shoal depths, * * * and the whole is covered on the seaward slope with very heavy rock ranging in weight from 6 to 16 tons per piece."

These extracts indicate that the usual resources of jetties built out from the shore by the aid of trestles and laid on mattresses, supplemented by dredging, are at least very unsatisfactory, if not impractical, at this locality, because of the activity of the teredo and the waves, so that the method has been modified to conform more closely to that proposed in the recent paper* by H. C. Ripley, M. Am. Soc. C. E.

Notwithstanding the great skill and ingenuity expended in the construction and maintenance of this trestle, more than 6 miles in length, and the excellent system of operation, the physical conditions are such as to retard its advance to such an extent that it seems

* "How to Build a Stone Jetty on a Sand Bottom in the Open Sea," *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 1040.

Mr. Haupt.

impossible to overtake the deposits of littoral drift which constitute the bar; for the last official report available at this writing, states the following facts: The estimated cost to secure a depth of 40 ft. was $3 715 000. This project was revised to raise the jetty to mid-tide level, in 1909, at a cost of $3 529 300, and to make it 25 ft. wide at that level. In some places it is in 39-ft. depths, thus requiring an enormous increase in the amount of rock. Between September 20th, 1910, and June 30th, 1911, the dredge worked on the bar 62 days, removing 212 080 cu. yd. The project is 85% completed.

"The survey of June shows the channel to have shifted about 2 500 ft. to the northwest, and the depths vary from 25 to 27½ ft., an increase of 1 ft. over last year.

"The life of the trestle is very uncertain. * * * It is believed that the contraction of the entrance by the north jetty may be followed by scouring along the north side of the south jetty (2 miles distant), and that for its maintenance two more groins should be provided, each about 500 ft. long. The total appropriations since 1902 aggregate $7 901 852.25. The outer 24-ft. contour has advanced some 3 000 ft. since 1902, and the outer depths of 50 ft. in that year had shoaled to 24 ft. by 1909. On the southerly side of the jetty the 24-ft. curve had advanced about 4 000 ft. within 2 years, or about twice the length of the jetty extension in the same time, and large deposits had been made in the throat of the entrance between the jetty and Peacock Spit."

The deductions from these statements are that the extension of the jetty has caused deep erosion at its outer end, requiring enormous quantities of rock to fill voids formerly occupied by sand in place; the rapid advance of the outer slope, accompanied by a shoaling of 26 ft. at the 4-fathom contour; the inability to overtake this advance by the jetty; the shifting of the channel some ½ mile to the northwest by the deposits of drift; the great excess of cost, amounting to more than $1 000 000 per ft. of depth gained; and the proposal to erect the second jetty extending out from Cape Disappointment for a distance of 2 miles, and at this same distance from the present jetty, but neither of them reaching to the crest of the bar by nearly a mile. The result of this must be to aggravate the seaward movement of the bar, while affording little or no protection for dredging.

It is also apparent that the report of 85% completion does not apply to the north jetty, nor to the ultimate extension of both, if this method is to be the main reliance for the improvement of this bar. The purpose of the south jetty, slightly convex to the channel, 4¼ miles in length, was to extend the protecting spit to a point abreast of Cape Disappointment and thus secure the co-operation of that headland, as a second jetty. This has resulted in disappointment, and any extensions of two jetties, it would seem, would give no promise of any different result, because there is no change in the general regimen

of the entrance, affecting the relative equilibrium of the flood and ebb currents. Mr. Haupt.

If the life of the piling, due to the teredo, is taken at the maximum limit of 20 months, then the entire structure subject to their attacks, must be rebuilt in that brief time, necessitating constant repairs and heavy expenses for maintenance. Moreover, the statement from actual experience that the dredge was able to work 62 days out of 283, or 22% of the time, covering the winter season, would give a reasonable assurance that it may be quite possible to work on the bar by creating an insular barrier on its outer slope, as a nucleus, sufficient to serve as a breakwater to protect a floating plant. From this a rock jetty could be extended shoreward to connect the deep-water areas on its outer and inner slopes, and to protect the crest from the littoral drift, as well as create a reaction from the impingement of the ample discharge on its concave face which would cut a channel more than 40 ft. deep automatically, as is shown by the existing channel at the base of the Point Ellice headland just within the throat of the entrance, which channel is about 8 miles long and more than 40 ft. deep, having a radius of 5 miles.

So confident is the writer that such a permanent, navigable channel could be readily obtained that, on June 9th, 1902, at the instance of the Senator from Oregon, he filed with the Secretary of War a proposal to guarantee such a channel for the sum of $2 500 000, which proposal was referred to a Special Board of Engineers for consideration and report. On October 14th, the Board gave a hearing to the officials of the Reaction Jetty Company (organized with a capital of $1 000 000 to execute the work), at which time a full discussion was submitted to the Board, all the members being present. On November 10th, 1902, the Board requested further information, and submitted twenty-three queries which were categorically answered on the 17th, and after several months of consideration, the report was filed with the Secretary of War, but its contents were not disclosed, and the Reaction Jetty Company was advised that it could ascertain the findings when the annual report of the Chief of Engineers was published. As that report has expunged all reference to the special provisions of the tender then submitted, and as 10 years have elapsed since it was made, and the recommendations of the Board, which were estimated to cost $2 260 000 for the extensions of the south jetty, at mid-tide level, a distance of 2½ miles, have been executed and for which appropriations approximating $8 000 000 have been set aside, it would seem appropriate that the terms of that proposal should be reviewed, as a matter of interest to the parties concerned, merely as an index to the early solution of this difficult but important problem of securing an open channel of ample capacity at reasonable cost without bar advance.

Mr. Haupt.

The principles involved have been tested on a practical scale at a condemned inlet on the Texas Coast, and notwithstanding the most serious physical as well as "metaphysical" obstacles, a detached jetty actually secured the full predicted depth of 20 ft. before the work was connected with the shores. Then it was seriously impaired, and the channel shoaled to 9 ft., requiring several years before equilibrium was restored. Thus assurance becomes doubly sure.

Briefly, the reaction jetty proposal reviewed the physical condition of the bar and its changes, as shown from official charts, stating that to obtain the 40-ft. channel would require the removal of some 30 000 000 cu. yd. which would be impracticable by dredging in the open sea with no protection works, as it would require some 30 years, if there were no littoral drift to maintain the supply, and would cost more than $6 000 000.

To control this drift and create an automatic channel, the company designed a permanent structure to create a zone of local activity across the bar, to arrest the drift on its convex face, being on the windward or "weather" side of the proposed crossing, and, at the same time, to cause a continuous reaction, with erosion and deepening on its concave face, by utilizing the forces and agencies of impact, head, reaction, concentration, gravity, and continuous deflection, whereby a sufficient amount of the potential energy of the affluent water is developed locally to produce deep scour and lateral transportation.

The old jetty, completed in 1895, which created the temporary 30-ft. depth, had caused the bar to shoal up at the rate of 3 500 000 cu. yd. per annum, and pushed it seaward about 1 mile, reducing depths of 60 ft. to 30 ft., and less.

To avoid this advance, the company proposed to make its work discontinuous, leaving a gap of about 2 miles between the outer end of the existing jetty and the inner end of the breakwater. The company also proposed to reverse the direction of the curvature of the breakwater, so as to place its cutting face toward the ebb currents and to receive them at first tangentially and, by gradual but continuous deflections, cause a constantly increasing pressure on the most vital part of the ebb for a distance of about 2 miles across the bar. Thus, too, the ingress of the flood tide would not be impaired, and the conditions of equilibrium would be changed in favor of the ebb stream.

Numerous other advantages, not incidental to straight jetties, were pointed out, time and expense were to be saved, and no risk was to be incurred by the Government, which was to be fully safeguarded by bonds, and pay only for work in place and depths secured.

Plans and cross-sections of the existing and proposed channels were submitted, with the form and dimensions of the proposed structure, closing with these words:

"As the proposed plan is novel and the location one of exceptional difficulty, this Company desires, if permitted, to accept all the responsibility for the success or failure of its plan, and will furnish such security as the Secretary of War may require to protect the United States from loss by reason of its failure. It will also satisfy the Secretary of its ability to carry out the contract to execute these plans as rapidly as the physical conditions will permit * * *. We believe that this proposition will be found to be 'the most economical and advantageous to the Government,'* since it is based upon the utilization of a part of the enormous energy now going to waste over the bar." Mr. Haupt.

After summing up the special advantage to be secured by the acceptance of the proposal, the Company pledged its good faith for its execution in the following words:

"If this Board, therefore, can see its way to recommend to the War Department the acceptance of this proposition for the relief of the Columbia River Basin, it is believed that it will perform a public service of great value to the country, and the Company will accept the trust and the responsibility of its execution in good faith, and will release the Government from any or all claims for royalties incidental to the use of its patents at this place."

The sequel to this, and similar tenders with like guaranties, indicated so plainly that the policy of past administrations was not to encourage the letting of contracts to extrinsic parties, however guaranteed, that the Reaction Jetty Company was dissolved.

The changes on the bar during the past 10 years have served to confirm the predictions made to the Board of 1902 as to the probable effects of the Government plan, and the elaborate report submitted at that time only alluded to the offer of the Company in a single paragraph as follows:†

"The Board knows of no plan for the improvement of this entrance that has not contemplated one or two jetties extending continuously seaward from the points of the entrance that are fixed in position naturally or artificially. The turbulence of the bar is such that operations from floating plant have never been seriously considered as practicable, and any work must be executed from a structure built out from the shore. Even the structure proposed by the Reaction Jetty Company, while nominally a detached breakwater, is in effect an extension of the old jetty, * * *."

This was the only reference, and the extension of the south jetty, as since constructed, was recommended, notwithstanding the following statements in the report:

"The Board cannot expect that an advance of the bar at this point will not follow the construction of the jetties and the removal of the large quantity of sand necessary to secure a 40-ft. channel. Such advances have invariably been found in all jetty harbors."

* Terms of the law authorizing the Secretary to make such contracts.

† Report, Chief of Engineers, U. S. A., 1902, p. 2305.

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But it is expected that:

"The waves and strong littoral currents have at this point their maximum effect, in retarding and counteracting the bar advance, and, unless the history of the past 60 years is misleading, that advance will be speedily checked and probably reversed with a return of the outer bar slope toward its present location."

These hopes do not appear to have been justified by the results in the later reports, and the question arises as to whether or not there may ever be a better solution than the two-jetty plan, which, it is conceded, "invariably" advances the bars in "all" cases. The answer may be suggested by the experience at Aransas Pass during the operation of the partial reaction breakwater, which not only prevented bar advance but caused a recession of the outer contours until the channel was cut entirely through, without injury to the structure, by a feeble diurnal tide. There are many other elements in this report of the Board of 1902-03 which are worthy of consideration, but space and time prevent the writer from mentioning them.

It is hoped that these suggestions may open the door to a broader consideration of the policies and possibilities of this nation for removing physical obstacles to international trade, and be of greater economic advantage to all people.

Some light is thrown on the status of the two-jetty system of surmounting ocean bars by the following extracts from a letter by Col. E. H. Ropes, a layman of New York, who was endeavoring to effect an entrance from the Gulf of Mexico to the bluffs across Corpus Christi Bay, Texas, addressed to Mr. Brewster Cameron, also a layman, interested in promoting the opening of the near-by entrance at Aransas Pass where three previous efforts had failed.*

As this letter expresses the opinion of one who had given his time and means to a project on which he was willing to stake his fortune, after careful investigation, it has a direct bearing on the subject as a practical demonstration of the futility of undertaking a maritime work without consideration of all the elements affecting the problem.

"The Plea for 'Ropes' Pass.'

"NEW YORK CITY, May 24th, 1894.

"MR. BREWSTER CAMERON,
"Baltimore, Md.

"DEAR SIR:— * * * If you are in a position to go ahead with Aransas Pass, I have no proposition to make. If you are not, or if you had considered the outcome of your efforts there so dubious as to incline you to consider a proposition from me I should make one.

* This letter is a part of the posthumous correspondence left by Mr. Cameron who lost his life by slipping into the Niagara Rapids above the Falls after he had successfully reorganized the Aransas Pass Harbor Company and inaugurated the work on the plans of the reaction jetty which secured the requisite depths before the second jetty was built.

The whole thing turns upon the condition of your prospects for Aransas. Mr. Haupt.

"You are right in thinking I will be benefited by deep water at Aransas. It will be far more to me than the public supposes. I have, therefore, the best of reasons for wishing you success. * * *

"The objections you cite to Ropes' Pass are based upon wholly erroneous ideas concerning the scientific project, the commercial plan, and the financial methods. Of this I could satisfy you in a brief interview.

"1. The channel across Mustang Island, no where goes through land 30 feet high. Half the distance is through flats covered at high tide; the rest has an average height of 3 to 10 feet, * * * while a vast amount of dredging will be required at Aransas for dockage in any depth exceeding 12 feet. At Ropes' Pass the dredging for channel makes the dockage, and by reason of the shorter passage and greater depths the currents will be immensely more powerful to scour, thus saving much excavation.

"2. The dockage, being on both sides of the channel, is now completely land-locked. No harbor could be more so.

"3. We can also make it a free port * * * while Aransas will doubtless charge tolls.

"4. But the jetty work is the big item, as has been shown at every Texas point. Dredging in smooth water is child's play in comparison. Do you know that more than $9 000 000 has been spent on Texas jetties, and with no important permanent results? Ropes' Pass requires less jetty work than any proposed harbor on that coast.

"5. * * * I know more now than I did five years ago about Texas sand-bars. The official reports show that every prophecy concerning the results of work on each of the Texas bars has been negatived by experience. Latterly the Reviewing Boards have been denouncing such predictions as the product of 'ignorance' and as attempts to impose upon 'the credulity of mankind'! Engineers of the highest rank declare that no prediction can possibly be made of the effect of operations on those bars. All the strong language occurs in the official reports. Galveston was the immediate theme, but the principles apply equally well to all the Texas bars. The facts sicken me of all bar work on that coast. Why, they have been at work 24 years at Galveston, and at other points nearly as long. You know the results. Every bar, as soon as it is touched, commences to move seaward. That is the real trouble at the Brazos. The existing jetties end where deep water was when they were begun. The bar has been racing with the jetties, just as at Galveston.

"6. Ropes' Pass has great advantage over others in length of jetties, in size, depth, and nearness of internal basin, and in other important matters, but none counts as much as the fact that we have no bar to deal with. It is a steady slope all the way from beach to 90-ft. depth. We can figure exactly how much rock will be required to build our jetties to any given depth. No living man can do this at any other point.

"If the bars had only stood still at Galveston and Brazos, the jetties would now stand in deep water. Not only this, but no bar can form at Ropes' Pass until the whole shape of the coast has changed, because

Mr. Haupt.

our jetties will shut away the bar building forces from the channel, and it will take hundreds of years for the whole island to build out to these ends.

"Your impression that the conditions are 'more favorable' at Aransas than at Ropes' is natural, but erroneous."

(The remainder of the communication refers to financing the project, and asks for a conference.)

The sequel is well known, for the effort to dredge a cut across Mustang Island into Corpus Christi Bay merely made an indent in the beach, destroying the continuity of the littoral drift, which was driven into this artificial bight, and closed up the channel in the wake of the dredge which was imprisoned in its own grave. Had the leeward jetty been built, in advance, to a sufficient distance to intercept the drift, there might have been a fair chance of opening the cut through to the bay, and, with proper work, of maintaining it.

This work, therefore, was abandoned, while that at Aransas Pass, due to the new order of construction by building in from the outer end by a floating plant and on a concave trace, has automatically scoured out its own channel, more than 20 ft. in depth and 300 ft. in width, without bar advance, at a cost to the Government of less than one-fourth of its original estimate, and with a 14-in. diurnal tide. It is true that subsequently a second straight jetty was built, at a cost, including other superfluous works, of more than $1 000 000, but the immediate effect of connecting the "Haupt breakwater" with the shore of St. Joseph's Island was to shoal the 20-ft. channel to about 9 ft. and reduce the tidal energy to such an extent that nearly 4 years elapsed before the former 20-ft. channel was recovered. This experience, therefore, furnishes an excellent practical example of the removal of an ocean bar by a portion of a single barrier placed across its crest, without aid from dredging and without bar advance, at a very great economy in cost, and should commend itself to the Profession as worthy of acceptation in many other cases, notably at the mouth of the Columbia River, Grays Harbor, Coos Bay, etc., on the Pacific Coast, where the conditions are exceptionally favorable.

Gen. Chittenden.

H. M. CHITTENDEN, M. AM. SOC. C. E. (by letter).—Mr. Goodrich has permitted an unnecessary sensitiveness to the writer's rather jocular commentary on his ambitious vision of the future port of Los Angeles to give a false color to his estimate of what the writer said about that port. A careful perusal of the treatment of that subject, mainly in Section II of the paper, will show it to be substantially accurate, though lacking in detail, as was necessarily the case. Particularly inexcusable is the following remark by Mr. Goodrich: "Nothing but an obviously exaggerated newspaper report has been used in describing the port conditions at Los Angeles." The most superficial examination of the paper would show the absurdity of that

statement. As a matter of fact, all available literature, including Major Fries' admirable report, was used, and this was supplemented by a long list of questions sent to an official of the Port of Los Angeles asking for data on all phases of the question. Gen. Chittenden.

As to the newspaper report, which is what really annoys Mr. Goodrich, it came to hand just before the paper was finished. It fitted in so perfectly with the writer's treatment of "plans for the future" and seemed to be so much in detail and to bear so unmistakably Mr. Goodrich's authority, that the writer ventured, with some misgiving, it is true, to make use of it. If it committed him to any erroneous suggestions as to the future of Los Angeles which later data would have modified, he is, of course, only too glad to accept corrections.

The writer will not comment on Mr. Goodrich's references to estimates of cost of Los Angeles work, nor to administrative control, for they are so manifestly unjust that Mr. Goodrich must himself realize the fact if he will but compare his criticism with what the writer really said.

The comment on the writer's parenthetical reference to the Los Angeles and San Gabriel Rivers is a case of assuming that something was meant which was not said, and then criticising that something. The writer's description of the natural state of those streams is literally correct; but he did not anywhere say, or even imply, that these streams were a menace to the integrity of the harbor. The writer has carefully looked over his paper to see if it contains any hint of danger "of the inner harbor of Los Angeles becoming silted up by material carried in the floods" of these streams, and he can find none. All this, and the careful refutation of it, are purely the result of Mr. Goodrich's imagining what the writer did not say.

In short, Mr. Goodrich has permitted himself, most mistakenly, to class the writer as a "knocker" of the Port of Los Angeles. Yet a perusal of the paper will show it to be, not only wholly sympathetic in tone, but full of admiration for the wonderful work which the people of Los Angeles are doing.

The impartial reader of the paper cannot fail to observe that it endeavors to draw a just comparison among the leading ports of the coast as to their respective advantages and drawbacks. The partisans of each will naturally take exception as to their particular port, magnifying the advantages and minimizing the drawbacks; but, when all is said and done, the case for the principal United States ports on the Pacific Coast will stand substantially as summarized below:

San Diego.—Advantages: A magnificent natural harbor which, with a moderate degree of development, would accommodate the shipping of the world.

Drawbacks: A sharp restriction of hinterland (tributary commer-

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cial territory) by the national frontier immediately to the south and a mountain range on the east. The first natural outlet to the eastward is in the vicinity of Los Angeles.

Los Angeles.—Advantages: Commercially located where the great southern transcontinental routes, and also one from the interior, debouch from the mountains to the coast plain; strategic location for ocean terminal; extensive and productive near-by hinterland; good connection with the hinterland farther back; climate an asset of enormous value in development of country.

Drawbacks: Entire absence of an adequate natural port, and consequent necessity of creating one almost wholly by artificial means; cost of port development greatly enhanced by this fact.

San Francisco Bay.—Advantages: A perfect natural harbor of great magnitude, close to the sea; a hinterland which embraces the heart of California's agricultural and mineral producing regions—one of the richest in the world; on the line of the great routes north and south; in direct rail connection with all the region to the eastward; a commerce and development half a century old, with the immense force of customs and established lines of trade in its favor; most centrally located of all the ports for the commerce of the Pacific; important naval and military bases.

Portland.—Advantages: Unexcelled strategic location at great crossroads of north and south, and east and west routes; a productive local hinterland (agricultural) in the Willamette and other near-by valleys; immense timber resources; a water-grade route to the eastward—the only one from the Pacific Coast of the United States; consequently a vast natural interior hinterland.

Drawbacks: Long distance and defective natural channel to sea which discourages entrance of largest vessels—a drawback as yet only partly overcome.

Puget Sound.—Advantages: Almost perfect natural harbors, the principal defect being excess of depth; center of great timber producing region; center of great fish producing region; limited local agricultural hinterland; extensive agricultural hinterland back of the mountains; the natural emporium for the Alaska trade; well located for Oriental trade; important naval and military base.

Drawbacks: Walled off from natural hinterland to eastward by an immense range of mountains, the main passes being less than 75 miles from tide-water and more than 3 000 ft. high; crossed, however, by three railways, which give same terminal rates as to Portland.

A most striking illustration of the drawbacks above described of both the Lower Columbia and of Puget Sound was seen in the conditions which prevailed on January 7th and 8th, 1913. A fierce storm was causing wrecks and great loss of life at the mouth of the Columbia, while deep snow, avalanches, and (soon after) floods from melting

snow completely paralyzed rail traffic over the mountains back of Puget Sound, and caused considerable loss of life. Permanently to remove either the handicap of the Columbia Bar or the handicap of the Cascade Mountains will cost an enormous sum, yet both ends must be classed as within the range of practical engineering. Gen. Chittenden.

In the foregoing comparative estimate of advantages and drawbacks of Pacific Coast ports it will be noted that San Francisco stands first; and it seems to the writer that she will long continue to stand first, because of her unexcelled advantages. It has also become more and more apparent to him that, unless ports are close competitors in the same field—not distant competitors like Puget Sound and California ports—a mere superiority of port facilities cuts little figure in the matter of competition. Vessels are bound to go where the trade is, even if docks are inferior and charges high, and the writer does not believe that San Francisco or Seattle will take much trade from each other by reason of superior expenditures on docks and wharves; but, with two nearby ports like Seattle and Tacoma, both of which are equally well located, as far as local trade is concerned, and between which the vessel owner sees no choice except in the matter of conveniences and charges, a superior equipment in either might be decisive in its favor.

The writer will refer to one other point, that of port administration. Mr. Goodrich says that Los Angeles has corrected the undesirable condition in port administration prevailing at the time this paper was written. That is fortunate, but the writer does not believe that anything approaching an ideal system will ever be worked out as long as port administration is a function of city government and is thereby necessarily involved in city politics and subject to the vicissitudes of changing administration. That this is the view of sagacious observers on the ground is evident from the following extract,* received after the foregoing statement was written:

"Inefficiency and delay, bickering, obstruction, and even deadlocks will unquestionably be the rule rather than the exception so long as the present machinery for harbor development is relied on by the city. The concentration of power, authority and responsibility in one board, under which the legal, engineering and administrative departments would be placed, will probably have to come before much satisfactory progress is made on this all-important municipal project—the development of Los Angeles harbor."

In a most instructive pamphlet recently issued by the Commonwealth Club of California, Professor C. T. Wright, of the University of California, made some pertinent observations on port administration. The following are some of his conclusions:

"The organization of the harbor governing body should be simple and efficient. Its members should be few in number, with a tenure of

* From the *Los Angeles Tribune*, January 17th, 1913.

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office sufficiently long to permit of the formation and inauguration of a consistent policy.

"The appointment of a full new board almost always results in an entirely new policy. A new board usually rejects a great part of the programme outlined by its predecessors, and spends a large part of its time in preparatory work before it begins or pursues any real constructive programme; and this in turn is superseded by the plan of a succeeding board. The harbor governing body, if ideally constituted, would be a perpetual body, a certain proportion of whose members would retire each year. In other words, it ought not to be possible, or at least it ought not regularly to occur, that the entire board should retire at one time, or even that a majority of the board should do so.

"The harbor governing body should have full authority to carry out the work which it exists to do, subject of course to review by the courts or other higher authority. This power of the governing board should extend to all activities which directly concern shipping.

"A harbor governing body should represent a constituency of ample financial ability."

These are sound fundamental principles. While it is perfectly true as a rule that "What'er is best administered is best," and that a very efficient law inefficiently administered may give poor results while a defective law efficiently administered may give good results, still, in the long run, the best law will give the best results. The writer believes that the Washington State statute comes nearer to satisfying the above fundamentals laid down by Professor Wright than any other. The port organization is simple, consisting of three members, entirely free of State, county, or city politics, and responsible directly to the people of the Port District.

The harbor governing body (Port Commission) is a continuing body, the term of office being three years, and only one member being elected each year. No sudden change of policy, with its disturbing features, is possible, as is the case with the San Francisco Board, which is liable to change bodily with every change of State administration.

The powers of the Port Commission are extensive; its activities are directed to the single purpose of port development; it is almost supreme within its restricted sphere, and its authority satisfies perfectly Professor Wright's requirement that it be "both extensive and intensive."

The territorial extent of the constituency and also its financial ability are determined in the original organization of the district. The Port of Seattle, for example, was made co-terminous with King County, in which the City of Seattle is located. "Ample financial ability" does not necessarily mean large financial ability. A small port may have an organization and a financial ability "ample" for itself, which might be quite inadequate for a great port. The system is thoroughly elastic, and is applicable to all conditions, and the writer would commend a study of it by port authorities wherever changes are contemplated in port organizations.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1247

NOTES ON BRIDGEWORK.

By S. Vilar y Boy, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. J. C. Howalt, William P. Parker, and S. Vilar y Boy.

In submitting the following, the writer does not pretend to have solved any special points in engineering science. The paper contains only a re-statement of some of the problems which every-day practice has made of interest to him in the construction and maintenance of the Guatemala Railway, and this is the only reason for thinking that they may be of interest to those engaged in similar work.

For instance, the increase of the reaction on the intermediate support of a continuous beam for a double span is generally disregarded when computing trestlework; but, at the same time, and for the sake of stiffness, the stringers for trestlework are generally made continuous over two spans; therefore, the writer has considered it worth while to undertake the solution of this problem, in order to ascertain whether or not the general practice is approximate enough.

Of course, if such stringers are laid with broken joints, the total load on each bent is the same, but the partial reaction due to the stringer which is continuous on the bent considered is higher than that due to its fellow, therefore the load on such a bent is not symmetrical, and, in consequence, the foundations, whether mud-sills or piles, are likely to settle unevenly.

It is stated in the textbooks on applied mechanics that a continuous beam on three supports can be dealt with as two separate beams fixed at the intermediate point and simply supported at the ends.

In this assumption the writer has considered a concentrated load on a beam supported at one end and fixed at the other, and has worked out the solution of this problem in accordance with the general principles of the elastic theory, as follows:

If the beam be simply supported at the two ends, the reaction on A should be: $\frac{W\,b}{a+b}$, and the reaction on $B = \frac{W\,a}{a+b}$; but the effect

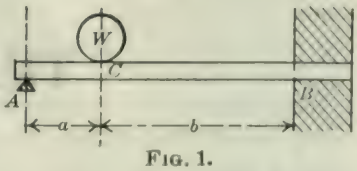


FIG. 1.

of the embedding or fixed support at B (Fig. 1) increases the reaction at this point and diminishes it at A, so that the reactions are:

$$\text{On } A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots X = \frac{W\,b}{a+b} - Z$$

$$\text{On } B \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots Y = \frac{W\,a}{a+b} + Z.$$

Note that the increase and decrease of the reactions on the two ends must be equal, so as to fulfill the condition, $X + Y = W$; that is to say, the sum of the two reactions must be equal to the total load.

The bending moment at C, referred to the support, A, is:

$$m = \left(\frac{W\,b}{a+b} - Z\right) a$$

and the bending moment, referred to B, is:

$$m' = \left(\frac{W\,a}{a+b} + Z\right) b - M,$$

M being the bending moment at the fixed end, B.

These two moments must be equal, as corresponding to the same section, C, so that:

$$\left(\frac{W\,b}{a+b} - Z\right) a = \left(\frac{W\,a}{a+b} + Z\right) b - M; \text{ from which,}$$

$$(a+b)\,Z = M \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Otherwise, the general equations of the neutral fiber applied on the segment, $A\ C$, should be:

$$E\,I\,\frac{d^2\,y}{d\,x^2} = \frac{W\,b}{a+b}\,x - Z\,x$$

$$E\,I\,\frac{d\,y}{d\,x} = \frac{W\,b}{a+b}\,\frac{x^2}{2} - Z\,\frac{x^2}{2} + C$$

$$E\,I\,y = \frac{W\,b}{a+b}\,\frac{x^3}{6} - Z\,\frac{x^3}{6} + C\,x$$

C being the unknown value of the constant in the first integral, the constant of the second integral being $= 0$, as representing the value of the deflection at the support, A.

The neutral fiber equations between C and B are:

$$E\,I\,\frac{d^2\,y'}{d\,x'^2} = \frac{W\,a}{a+b}\,x' + Z\,x' - M$$

$$E\,I\,\frac{d\,y'}{d\,x'} = \frac{W\,a}{a+b}\,\frac{x'^2}{2} + Z\,\frac{x'^2}{2} - M\,x'$$

$$E\,I\,y' = \frac{W\,a}{a+b}\,\frac{x'^3}{6} + Z\,\frac{x'^3}{6} - M\,\frac{x'^2}{2}$$

the constants in the two integrals being $= 0$ because B is a fixed end.

At the point, C, the two segments, $A\ C$ and $B\ C$, must be tangent to each other, and, therefore if $x = a$ in the first system of equations, and $x' = b$ in the second one, the two following conditions must be fulfilled:

$$y = y';\ \frac{d\,y}{d\,x} = -\frac{d\,y'}{d\,x'}$$

Therefore, the two conditions wanted are:

$$\frac{W\,b}{a+b}\,\frac{a^2}{2} - Z\,\frac{a^2}{2} + C = -\left(\frac{W\,a}{a+b}\,\frac{b^2}{2} + Z\,\frac{b^2}{2} - M\,b\right)$$

$$\frac{W\,b}{a+b}\,\frac{a^3}{6} - Z\,\frac{a^3}{6} + C\,a = \frac{W\,a}{a+b}\,\frac{b^3}{6} + Z\,\frac{b^3}{6} - M\,\frac{b^2}{2}$$

which can be reduced to:

$$\frac{W\,a\,b}{2} - Z\,\frac{a^2-b^2}{2} = M\,b - C$$

$$\frac{W\,a\,b}{6}(a-b) - Z\,\frac{a^3+b^3}{6} = -M\,b^2 - C\,a$$

The foregoing equations, together with Equation 1, will give the system:

$$(a+b)\,Z = M$$

$$\frac{W\,a\,b}{2} - Z\,\frac{a^2-b^2}{2} = M\,b - C$$

$$\frac{W\,a\,b}{6}(a-b) - Z\,\frac{a^3+b^3}{6} = -M\,\frac{b^2}{2} - C\,a$$

from which, after substituting for M its value as in Equation 1, and eliminating C, results:

$$Z = \frac{W\,a\,b\,(2\,a + b)}{2\,(a + b)^3},$$

and therefore

$$M = \frac{W\,a\,b\,(2\,a + b)}{2\,(a + b)^2}.$$

Call the total span l, and:

$$Z = \frac{W\,a\,b\,(l + a)}{2\,l^3}; \quad M = \frac{W\,a\,b\,(l + a)}{2\,l^2}.$$

The following is an application of the foregoing theory to the "Mogul" engines of the Guatemala Railway (3-ft. gauge) on the pile bridges of that road:

According to the actual conditions, the loading diagram for the maximum reaction on the intermediate support is as shown in Fig. 2.

The total load on B is equal to the sum of the two reactions due to the two 11-ton driver axles plus the 12-ton axle which bears directly on the support.

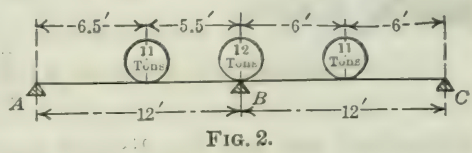


FIG. 2.

The reaction on B due to the left-hand span

$$= \frac{W\,a}{l} + \frac{W\,a\,b\,(l + a)}{2\,l^3} = 5.96 + 2.10 = \ldots 8.06 \text{ tons.}$$

The reaction on B due to the right-hand span

$$= \frac{W'\,a'}{l} + \frac{W'\,a'\,b'\,(l + a')}{2\,l^3} = 5.5 + 2.06 = 7.56 \text{ "}$$

Total.....	15.62 tons.
Axle bearing directly on B.....	12.00 "
Total reaction on B.....	27.62 tons.

Whereas, disregarding the continuity of the beam, the total reaction on the same point should be 23.46 tons.

Therefore, the increase due to the continuity is 18 per cent.

As for the bending moment, it is not worth while to take into consideration the continuity of stringers in order to calculate their size, because their strength is governed somewhat by the horizontal shearing along the neutral fiber, and the equation of moments for

the continuous beams would give too small a size to resist the horizontal shearing.

The following example will illustrate this:

With the same "Mogul" engine, the loading diagram for the maximum bending moment is as shown by Fig. 3.

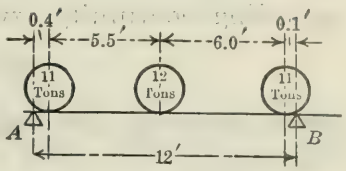


FIG. 3.

Disregarding the continuity:

The reaction at A ..... = 16.8 tons.
The reaction at B ..... = 17.2 tons.
The maximum bending moment ..... = 38.7 ft-tons.
The maximum bending moment under each rail .... = 19.3 ft-tons.
" " " " " " " .... = 232 in-tons.
The maximum shear under each rail ..... = 8.60 tons.

Assuming the cross-bending stress = 1 000 lb. per sq. in., that is, 0.5 tons, $\frac{I}{Z}$ (section modulus) = 464, which practically corresponds to two 6 by 16-in. pieces.

Continuous Beam.—

Left-hand axle	$Z = 0.18$ tons;	$M = 2.16$ ft-tons.
Middle axle	$Z = 2.23$ "	$M = 26.76$ "
Right-hand axle	$Z = 0.09$ "	$M = 1.08$ "
Totals	$Z = 2.50$ tons;	$M = 30.00$ ft-tons.

Therefore, the total reaction at $B = 17.2 + 2.5 = 19.7$ tons.

The maximum shear under each rail = 9.85 tons.

The maximum bending moment under each rail = 15.00 ft-tons = 180 in-tons.

If the cross-bending stress = 0.5 tons, $\frac{I}{Z} = 360$, which corresponds to two 4 by 16-in. pieces.

Longitudinal Shearing Strain.—

$$H = \frac{3}{2}\frac{\text{Maximum vertical shear}}{\text{Cross-section of beam}}$$

For two pieces 6 by 16 in. (disregarding continuity)

$$H = \frac{3}{2}\frac{8.6}{12 \times 16} = 0.067 \text{ tons} = 134 \text{ lb. per sq. in.}$$

For two pieces 4 by 16 in. (continuous beam)

$$H = \frac{3}{2}\frac{9.85}{8 \times 16} = 0.115 \text{ tons} = 230 \text{ lb. per sq. in.}$$

Whereas, the actual reaction, 9.85 tons, due to the continuous beam, applied to the size deduced from the bending moment, disregarding continuity (two pieces, 6 by 16 in.), would give:

$$H = 154 \text{ lb. per sq. in.}$$

which, although too high, is much more reasonable than two pieces, 4 by 16 in.

In the foregoing calculations, for the sake of simplicity, the writer has not included the dead loads; and, as he has referred to longitudinal shearing, he will point out the discrepancy between the safe shearing stress with the grain, as recommended by the Association of Railway Superintendents of Bridges and Buildings and the safe uniform loads on beams as per the tables reprinted from the *Proceedings* of the same Association.

These tables are computed, as explained in the preface of the pamphlet, assuming that: "the working stress in shearing is one-twentieth of that in cross-breaking."

To realize this, take for instance the tabular figures for a short-leaf yellow pine beam, 1 in. wide, 16 in. deep, on a 12-ft. span.

The foregoing formula, $S = \frac{3}{2}\frac{V}{b\,d}$, or $V = \frac{2}{3}\,b\,d\,S$, if $S = \frac{1\,000}{20}$ $= 50$, $b = 1$ in., $d = 16$ in., and noting that the total load is equal to twice the end reaction for uniform loads, gives:

$$W = \frac{4\,d\,s}{3} = \frac{4 \times 16 \times 50}{3} = 1\,070 \text{ lb.}$$

Therefore the tabular values correspond to a safe shearing stress of 50 lb. per sq. in., whereas the value generally accepted in ordinary practice is 100 lb. per sq. in., and therefore the safe load on the foregoing beam should be 2 140 lb. or, say, twice as much as shown in the table.

Therefore the assumption of "working shearing stress = one-twentieth of allowable extreme fiber stress" does not agree with actual practice. Doubtless this error came through overlooking these two facts:

The working cross-breaking stresses recommended by the Association of Railway Superintendents refer to the ultimate extreme fiber strain as an average between the ultimate endwise compressive and tensile strengths.

The ultimate cross-bending strains, as shown in the United States Forestry Department tables, agree fairly well with the values of R deduced from the well-known formula:

$$\text{Bending moment} = R \frac{I}{Z}$$

applied to a beam, 1 in. square, and of 1 ft. span, supported at each end, loaded with a center breaking load, W, given by actual tests.

The writer has not at hand the center breaking loads from the Forestry Department tests, but has come to this conclusion by using the values given in Trautwine's "Pocket-Book."

In the latter case it is a matter of fact that, for white pine and similar material, the ratio between shearing with the grain and cross-bending (ultimate) is about one-twentieth.

Another important point in bridge building is the depth to be given to a girder to stand a certain strain, which, as far as the writer knows, has never had a definite solution. Experience shows it to be from one-ninth to one-twelfth of the span, which is not definite. At the same time, experience shows the maximum allowable deflection of girders under good conditions, and from this the depth can be deduced as follows:

The value of the bending moment due to the actual axle loads being nearer to the value for the equivalent uniform load than to the moment due to a single concentrated load, we may refer the actual deflection of the girder to the formula for uniform loads.

For the sake of accuracy, however, the total uniformly distributed load must be computed so that the maximum bending moment it produces shall be equal to the maximum bending moment due to the actual loading diagram.

Call M the actual maximum bending moment, l the span, and W the total uniform load:

$$M = \frac{W\,l}{8}, \text{ and therefore } W = \frac{8\,M}{l} \quad \ldots\ldots\ldots\ldots (2)$$

The deflection is given by the formula:

$$f = \frac{W\,l^3}{77 \times E \times I}, \text{ and } \frac{1}{77} = \frac{5}{384}$$

From which,

$$I = \frac{W\,l^3}{77 \times E \times f} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Otherwise, call S the permissible unit stress, F the flange area, and d the depth of the girder:

$$M = S \times F \times d.$$

At the same time, assuming, as in regular practice, that the bending moment is entirely resisted by the flange area:

$$I = \frac{F \times d^2}{2}, \text{ and } F = \frac{2\,I}{d^2}, \text{ and therefore:}$$

$$M = \frac{S \times 2 \times I}{d^2} \times d = \frac{2\,S\,I}{d}, \text{ and } d = \frac{2\,S\,I}{M} \ldots\ldots\ldots\ldots (4)$$

From Equations 2 and 3,

$$I = \frac{\frac{8\,M}{l} \times l^3}{77 \times E \times f} = \frac{8\,M\,l^2}{77\,E \times f}, \text{ and substituting this value in Equation 4,}$$

$$d = \frac{2\,S \times 8\,M\,l^2}{77 \times E\,f\,M} = \frac{2 \times 8 \times S\,l^2}{77 \times E \times f}, \text{ in which, of course, the span, } l, \text{ is expressed in inches; but, if the span be measured in feet:}$$

$$d \text{ (inches)} = \frac{2 \times 8 \times 144 \times S \times l^2 \text{ (feet)}}{77 \times E \times f \text{ (inches)}}.$$

Assuming $E = 29\,500\,000$ lb., and $S = 10\,000$ lb.,

$$d = \frac{2 \times 8 \times 144 \times 10 \times l^2 \text{ (feet)}}{77 \times 295 \times 100 \times f} = 1.014\,\frac{l^2 \text{ (feet)}}{100 \times f}.$$

As for the value of f, although it might go up to the limit of 1 in. in a 100-ft. span, the writer finds, after working out the depths of the plate-girder spans on this road from 25 to 85 ft., that the results of the foregoing formula (assuming a deflection of $\frac{7}{8}$ in. in a 100-ft. span) agree with the actual dimensions within 2 or 3 in. in most cases.

The following is an example: In the 80-ft. girder span over "Los Andes" River on the Guatemala Railway, $l = 80$ ft. The deflection should be $0.875 \times 0.8 = 0.7$ in. in order to keep the standard of $\frac{7}{8} = 0.875$ in. per 100 ft. of span.

In this case

$$d = 1.014\,\frac{6\,400}{100 \times 0.7} = \frac{1.014 \times 6\,400}{70} = 92.7 \text{ in.}$$

The actual depth of the girder as built by the Baltimore Bridge Company = 90 in.

DISCUSSION

W. J. C. HOWALT, ASSOC. M. AM. SOC. C. E. (by letter).—The author says: Mr. Howalt.

"It is stated in the textbooks on applied mechanics that a continuous beam on three supports can be dealt with as two separate beams fixed at the intermediate point and simply supported at the ends."

The foregoing statement holds good only when the following conditions are fulfilled:

1. The two spaces must be equal.
2. All three supports must be on a straight line.
3. Cross-sections of the beam symmetrical around the center support must be equal.
4. The loads must be arranged symmetrically around the center support.

Conditions 1 and 3 are fulfilled in the author's case. Condition 2 is assumed to exist, but, when settlement is feared, it is quite advisable to consider what effect this may have. Condition 4 is not fulfilled at all, and renders the whole deduction void.

Apply a load, as in Fig. 1, and call the supports A, B, and C, as indicated in Fig. 2. The reaction at B, as found by the author, happens to be correct. The reaction at A will be $\frac{W\,b}{a+b} - \frac{1}{2}\,Z$, and at C $= -\frac{1}{2}\,Z$; this, being an uplift, must be counteracted by anchorage.

On page 242 the author accounts for the total external load in the reactions at A and B (his equation, $X + Y = W$). It must be assumed, therefore, that no anchorage is applied at the C end of the beam, but this, in turn, will change the continuous beam to a simple one supported at A and B, with an overhanging end on which no load acts, and for which the formulas are not applicable.

Now look at Fig. 3, which is supposed to be a diagram showing the loading for maximum bending moment for both simple and continuous beams; in neither case is this diagram correct.

In the first case assume the train coming in at B and stopped when the 12-ton axle is 4.69 ft. from B. We will then get a maximum moment of 42.07 ft.-tons against the author's "maximum" moment of 38.7 ft.-tons.

Regarding the second case: The writer will only refer to Fig. 2. This position of the load gives a moment at $B = 49.92$ ft-tons against the author's "maximum moment of 30.00 ft-tons. In both cases the

Mr. Howalt. formula for moment, M, on page 244, was used, which formula, by the way, is wrong.

The author makes no distinction between reaction and shear. With loadings as in Fig. 3 and with continuous beams, the maximum reaction is 19.7 tons, but the maximum shear is only 18.55 tons. He should explain why he uses the same position of the load for finding both maximum moment and maximum shear.

Mr. Parker. WILLIAM P. PARKER, M. AM. SOC. C. E. (by letter).—The author's treatment is on the assumption that a continuous beam on three points of support can be dealt with as two separate beams fixed at the middle point and simply supported at the ends.

This assumption is incorrect, as a load on either span affects the reactions on all three supports.

For any load, W (Fig. 4), the sum of the reactions at 1 and 2 is greater than W, while the reaction at 3 is negative. The three reactions, $R_1 + R_2 + R_3$, of course, are equal to W.

The purpose of the investigation seems to be primarily to find the reactions at R_2, the intermediate support, for any load or series of loads. By the method in the paper, the results give a much less reaction than the correct one. The writer, for his work in reinforced concrete design, uses the curves in Fig. 4, which give directly the reaction at all three points of support for a concentrated load in any position and the results from a series of loads can be combined arithmetically to give the resultant reactions. With the reactions found, it is easy to ascertain the bending moment and shear at any point on the beam, and locate points of contraflexure.

Using the notation on Fig. 4: From the theorem of three moments, for any load in the span, 1-2, W produces:

$$\begin{aligned} \text{Moment } M_1 &= 0 \\ \text{Moment } M_2 &= -\tfrac{1}{4}\, wl\,(a - a^3) \\ \text{Moment } M_3 &= 0 \end{aligned}$$

Taking the center of moments at the different supports, and expressing the given moment, the algebraic sum of the reaction and loads multiplied by the arms gives three equations. These, together with the equation, $R_1 + R_2 + R_3 = W$, make it possible to solve for the values of the reaction, as follows:

$$\begin{aligned} R_1 &= +\, wb - \tfrac{1}{4}\, w\,(a - a^3) \\ R_2 &= +\, wa + \tfrac{1}{2}\, w\,(a - a^3) \\ R_3 &= -\tfrac{1}{4}\, w\,(a - a^3) \end{aligned}$$

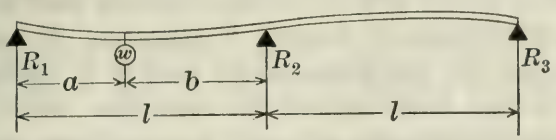
These three equations were used in laying out the curves in Fig. 4.

The results from the use of the curves are readily checked, for, having found the reactions for a given W,

$$R_1 + R_2 + R_3 = W.$$

Mr. Parker.

CONTINUOUS GIRDER OVER 3 POINTS OF SUPPORT



$M_2 = -\frac{1}{4}\, wl\,(a - a^3)$

$R_1 = +wb - \frac{1}{4}\, w(a - a^3)$, $R_2 = +wa + \frac{1}{2} w\,(a - a^3)$, $R_3 = -\frac{1}{4}\, w(a - a^3)$

Diagram gives reactions at Supports R_1, R_2 & R_3 for a Load "w" at distance "a" from Support R_1. Lower horizontal line represents different values of "a". Where vertical from given value intersects the different curves will give directly % of "w" which goes to each of the Supports.

For check $R_1 + R_2 + R_3 = 100\%$

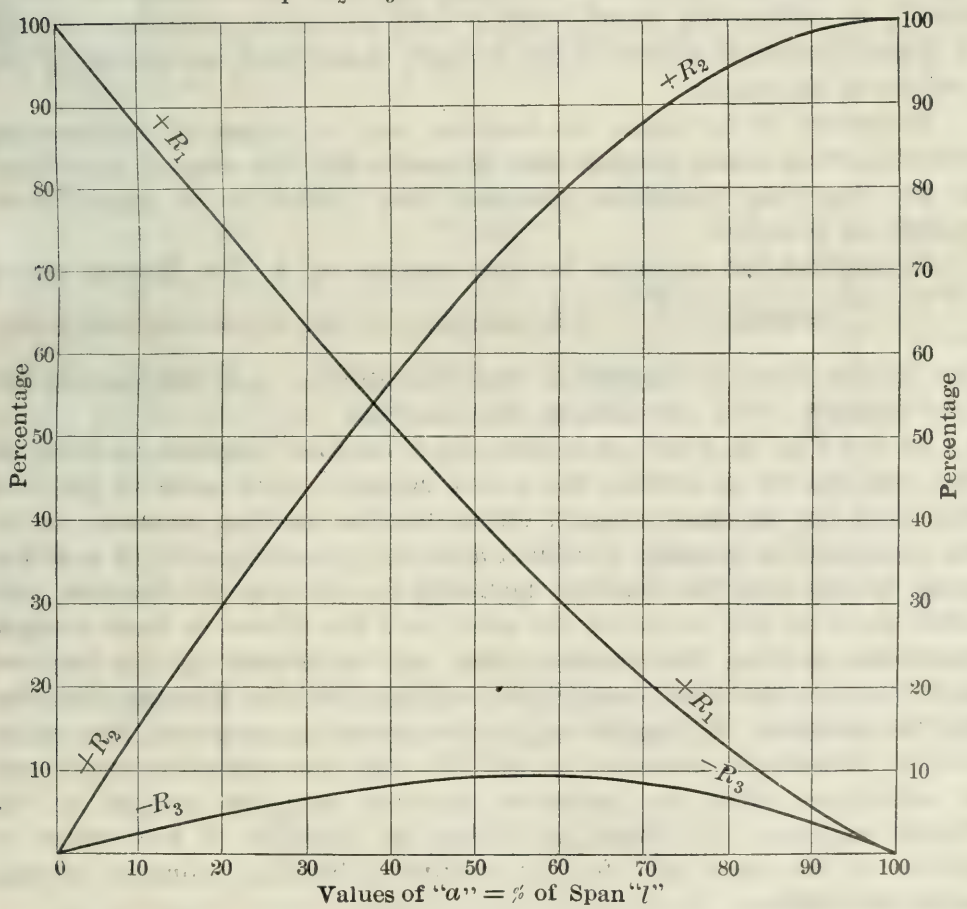


FIG. 4.

Mr. Vilar y Boy.

S. VILAR Y BOY, ASSOC. M. AM. SOC. C. E. (by letter).—Regarding the four conditions required to deal with a continuous beam as though it were two independent beams fixed at the intermediate support, Conditions 1 and 3 are of course fulfilled and also Condition 2, because, when settlement is feared, a structure is not supported at different levels, but on the same level; and it is under the condition of the same level that loads act on the structure, which will settle unevenly if the foundations are not well proportioned to the loads they are to sustain. This is what must be avoided; therefore, it cannot be considered as an actual working condition.

As for Condition 4, it is not mathematically fulfilled, but might be accepted as approximate enough, because the position of the engine for the maximum reaction on the intermediate support agrees very nearly with it, for which the writer refers to Fig. 2 and to Article 15 of "Railroad Construction," by W. L. Webb, M. Am. Soc. C. E., in regard to the possibility of assuming uniform axle loads, uniformly spaced, in computing trestle works, with results approximate enough in actual practice, which is the writer's standpoint, as stated in the preface of the paper.

Moreover, it is usual, in working out solutions of engineering problems, that some assumptions be made, for the sake of simplicity in the resulting formulas, provided they prove to be approximate enough in practice.

In quoting the equation for the reaction at A, Mr. Howalt shows $\frac{W b}{a+b} - \frac{Z}{2}$ instead of $\frac{W b}{a+b} - Z$, according to the writer's development; that is the point in discussion, and the uplift, $-Z$, has been taken into account when calculating the reaction.

As for Fig. 3, which was used as a loading diagram to compute the cross-section of beams, the writer accepted it in spite of the fact that it is not absolutely exact, either for the bending moment or for the end reaction, because, if refined accuracy is not required, it is rather usual to compute the bending moments by placing the heaviest axle loads about in the center of the span, and the others in their relative positions. As for the reactions, they are calculated for the heaviest total load on the beam, and, after stating that the formula obtained for the moment, M, ought to be disregarded in practice, the writer did not consider it necessary to go fully into the matter, as everybody is acquainted with the influence diagrams and the method of the French engineer, M. Barré, of fixing the position of the center of gravity of the axle loads for the maximum bending moment on end-supported beams.

The foregoing assumption might be checked as follows:

Let $M = 38.7$ tons, as accepted in the writer's calculations, so that $M = 19.3$ tons under each rail $= 232$ in-tons $= 0.5$ tons (permissible

stress) $\times \frac{I}{Z}$ (section modulus). Therefore, $\frac{I}{Z} = 464$, which practically corresponds to two 6 by 16-in. pieces, the section modulus of which is 512. Mr. Vilar y Boy.

Using the bending moment, $M = 42.07$ tons, as worked out by Mr. Howalt, M, under each rail $= 21.03$ ft-tons $= 252.36$ in-tons, and assuming as before that $S = 0.5$ tons, $\frac{I}{Z} = 504.72$, which also corresponds to two 6 by 16-in. pieces.

Therefore, the writer's purpose being only the comparison of the resulting cross-sections for the same loading diagram, both in the case of the end-supported beam and of the one fixed-end beam, the use of Fig. 3 does not appear to be unreasonable.

The value of $M = 30$ ft-tons does not apply to Fig. 2, as Mr. Howalt appears to think, but to Fig. 3, which might be ascertained by checking up the arithmetical calculations, so that really it cannot be compared to the value, $M = 49.92$ ft-tons, obtained by Mr. Howalt for Fig. 2, even if such value be correct.

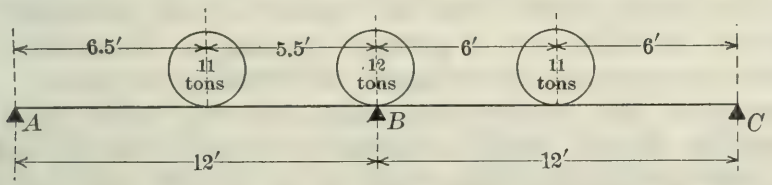


FIG. 5.

The writer does not like to insist on trifles, but his attention was called to the fact that the bending moment for a continuous beam under concentrated loads is higher than for an end-supported beam covering the same span at the rate of 49.92 ft-tons, as against 42.07 ft-tons (both figures calculated by Mr. Howalt), and although the matter has been explained quite definitely by Mansfield Merriman, M. Am. Soc. C. E.,* the writer checked up the foregoing figures as in Fig. 5.

The central load of 12 tons being disregarded, as it bears directly on B, and, therefore, produces no deflection, the theorem of three moments for concentrated loads might be applied to the two 11-ton axles, and therefore:

$$M' l' + 2 M'' (l' + l'') + M''' l'' = -P' l'^2 (K - K^3) - P'' l''^2 (2 K - 3 K^2 + K^3).$$

As the beam is simply supported at the ends, $M' = M''' = 0$. Also, $l' = l'' = 12$ ft., $K = 0.54$ for the first span, and $K = 0.5$ for the second span.

* "Mechanics of Materials," Chapter on "Comparison of Beams."

Mr. Vilar y Boy.

Then:

$$48\,M'' = -11 \times 144\,(0.54 - 0.16) - 11 \times 144\,(1 - 0.75 + 0.125)$$

$$48\,M'' = -11 \times 144\,(0.54 - 0.16 + 1 - 0.75 + 0.125)$$

$$= -11 \times 144 \times 0.755 = -1\,195.92\text{; and } M'' = -\frac{1\,195.92}{48}$$

$$= -24.91 \text{ ft-tons, as against 49.92 given by Mr. Howalt.}$$

Dealing with the continuous beam as though it were two separate beams fixed at the intermediate support, and applying the writer's formula for M, the results are as follows:

For the first span = 25.2 ft-tons;
For the second span = 24.7 ft-tons;

as against the 24.91 ft-tons given by the theorem of three moments, whereas Mr. Howalt's figures are about twice as great.

As the obtained value for the reaction at B "happens to be correct," and the value of M, according to the same method, also happens to be very close to the exact figures, and, moreover, as the whole deduction is in accordance with the general principles of the theory of flexure, the writer thinks that his method is not altogether wrong, although he considers that in practice it must be disregarded in computing the cross-sections of stringers, as stated in the paper.

Referring to Mr. Parker's discussion, the only point to be decided is whether or not the results obtained by considering the continuous beam as two independent beams, fixed at the middle point, are approximate enough in practice. Of course, the deduction on the basis of the theorem of three moments is mathematically exact and excludes any assumption, and, furthermore, Mr. Parker's formulas are so handy that had the writer known of them, he would have accepted and used them instead of working out the solution of the same problem in a different way.

At the same time, Mr. Parker's conclusion as to the method in the paper giving a much less reaction at R_2 than the correct one, is probably due to the fact that the percentage shown by the curves in Fig. 4 refers to the actual load on the beam, whereas the percentage shown in the paper refers to the calculated reaction without regard to the continuity of the beam.

The figures obtained by applying Mr. Parker's formulas to Fig. 2 will prove it:

$R_2 = wa + \frac{1}{2} w\,(a - a^3)$ gives the following results:

For the left-hand load,	$R_2 =$	8.03 as against	8.06 shown in the paper.
For the right-hand "	$R'_2 =$	7.59 " "	7.56 " " " "
Load resting on B.	$R''_2 =$	12.00 =	12.00 " " " "
Total reaction at B,		27.62 =	27.62 shown in the paper.

Mr. Vilar y Boy.

In order to prove that the foregoing result is not a mere coincidence, the writer tried to compare both formulas (Mr. Parker's and his own) from a theoretical and abstract standpoint, as follows:

To avoid any misunderstanding, the symbols, a and b, in the writer's formula will be expressed in Greek letters, α and β, because in such formula a and b represent the actual dimensions, in feet, whereas in Mr. Parker's they are fractional factors, so that the actual lengths of the two stretches of beam are al and bl, respectively, that is to say, $\alpha = al$ and $\beta = bl$, and as $\alpha + \beta$ must be equal to the total span, l, it follows that $a + b = 1$.

Therefore, the writer's formula:

$$R_2 = \frac{W\alpha}{l} + \frac{W\alpha\beta\,(l+\alpha)}{2\,l^3}, \text{ will be easily reduced to:}$$

$$R_2 = W\,a + \frac{W \times al \times bl\,(l+al)}{2\,l^3} = W\,a + \frac{W\,l^3 \times ab\,(1+a)}{2\,l^3}, \text{ or,}$$

$$R_2 = W\,a + \frac{W}{2} \times ab\,(1+a), \text{ but } b = 1 - a, \text{ so that}$$

$$R_2 = W\,a + \frac{W}{2} \times a\,(1-a)\,(1+a) = W\,a + \frac{W}{2} \times a\,(1-a^2)$$

$$R_2 = W\,a + \frac{W}{2}\,(a - a^3)$$

which is identical with the resulting formula on the basis of the theorem of three moments, this being why the writer believes that Mr. Parker's assertion, about the different results obtained from his and the writer's formula, is due to a misunderstanding.

Similarly, the writer's formula for R_1 might be reduced to:

$$R_1 = w\,b - \frac{1}{2}\,w\,(a - a^3) \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

against $R_1 = wb - \frac{1}{4}\,w\,(a - a^3)$ from the theorem of three moments, the difference between these two values being $-\frac{1}{4}\,w\,(a - a^3)$, the maximum of the absolute value of which (for a constant load, w) corresponds to the maximum value of $(a - a^3)$, the first derivative of which:

$1 - 3\,a^2$ equalized to 0, that is to say, $1 - 3\,a^2 = 0$, solved for a leads to $a = \sqrt{\frac{1}{3}} = \sqrt{0.3333} = 0.58$, which corresponds to a maximum of the function, as the second derivative is a negative expression.

Therefore, the maximum absolute value of $\frac{1}{4}\,w\,(a - a^3)$ should be:

$\frac{1}{4}\,w\,(0.58 - 0.20) = \frac{1}{4}\,w \times 0.38 = 0.095\,w$, or, in other words, the

Mr. Villar y Boy.

maximum error in the value of R_1, due to the writer's assumption, should be 9½% of the load, w, in case only one span is loaded, as shown in Fig. 4.

If the two spans are loaded, as in Fig. 6, the reaction at A should be as follows:

$$\text{On account of the load, } W, \quad r_1 = W\,b - \frac{1}{4}\,W\,(a - a^3)$$

$$\text{“ “ “ “ “ , } W', \quad r_1' = (\quad) - \frac{1}{4}\,W'\,(a' - a'^3)$$

and, therefore, the total reaction at A would be:

$$R_1 = W\,b - \frac{1}{4}\,W\,(a - a^3) - \frac{1}{4}\,W'\,(a' - a'^3)$$

and, if $W = W'$, $a = a'$, $b = b'$, or, in other words, if the spans be symmetrically loaded: $R_1 = w\,b - \frac{1}{2}\,w\,(a - a^3)$, which is identical with Equation (4), so that if the axle loads are uniform and uniformly spaced, which is the case of the engine drivers in the standard loading diagram, both formulas (Mr. Parker's and the writer's) should lead to exactly the same values.

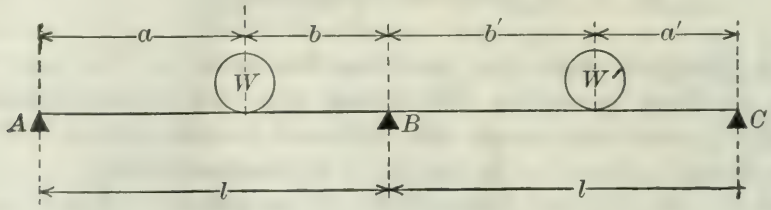


FIG. 6.

As for the actual axle loads, the conditions prove to be so close to symmetry that the difference between the results of both formulas may be disregarded in practice.

Referring again to Fig. 2:

$a = 0.54$, $b = 0.46$, $a' = 0.5$, $b' = 0.5$, $W = W' = 11$ tons, so that by using Mr. Parker's method:

$$R_1 = 11 \times 0.46 - \frac{1}{4}\,11\,(0.54 - 0.16) - \frac{1}{4}\,11\,(0.5 - 0.125)$$

$$= 11\,(0.46 - 0.095 - 0.094) = 11 \times 0.271 = 2.981 \text{ tons.}$$

Whereas, the writer's method gives:

$$R_1 = \frac{11 \times 5.5}{12} - \frac{11 \times 6.5 \times 5.5 \times 18.5}{3\,456} = 11 \times 0.46 - 11 \times 0.191$$

$$= 11 \times 0.269 = 2.959 \text{ tons.}$$

The error is $2.981 - 2.959 = 0.022$ tons, that is to say, less than 1% of the exact value, which is within the error of the slide-rule, the use of which is generally accepted.

Mr. Vilar y Boy.

Reviewing the whole discussion, the following conclusions might be obtained:

First.—As for the value of the reaction at the intermediate support (which was the writer's primary purpose), the assumption of a continuous beam being equivalent to two separate beams fixed at the intermediate support proves to be absolutely exact in any case.

Second.—As for the reaction at the ends and the bending moment, the above assumption is absolutely correct in the case of the two spans being symmetrically loaded, and approximate enough in practice for the actual loading diagrams.

Third.—As for the reaction at the ends and the bending moments, the assumption should be incorrect in the case of only one span being loaded, which is not the actual loading diagram in the problem submitted by the writer.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1248

THE STRENGTH OF COLUMNS.

By W. E. Lilly, Esq.*

With Discussion by Messrs. Edward Godfrey, J. O. Eckersley, Henry S. Prichard, J. S. Branne, and W. E. Lilly.

Of recent years many data have been accumulated from the published results of tests of the strength of columns and struts. The application of these tests to the various cases that occur in the design of columns, however, gives rise to some difficulties, owing principally to a want of some general method by which the experimental data may be co-ordinated. For instance, from the individual tests on braced struts, the proportions to be adopted for bracing can be approximately determined, but no rules based on these tests have been formulated which are generally applicable.

In this paper it is proposed to consider briefly the derivation of the Rankine-Gordon formula and the writer's formula for columns, and then compare some of the values obtained from tests on columns with those derived from the formulas applicable to the particular cases considered. Some criticism on column formulas and their application to the design of struts will then be given, and some suggestions offered as to the lines on which the co-ordination of experimental work in this direction should proceed.

A bibliography of the literature of the subject is given in the appendix, together with titles of papers by the writer referred to in the text.

At different times various formulas have been put forward to determine the strength of columns; of these, the writer considers the

* Lecturer in Engineering, University of Dublin.

Rankine-Gordon formula to be the best. The following quotation from Mansfield Merriman,* M. Am. Soc. C. E., also supports the writer's opinion:

"The theoretic basis of Rankine's formula seems far more satisfactory than that of any other which has been proposed for the discussion of such columns as are used in engineering practice."

This formula is of a form proposed by Tredgold and afterward revised by Gordon, who determined the approximate value of the constants; it was then modified by Rankine, who substituted the value of the radius of gyration for the diameter of the cross-section. The formula only applies to solid columns, and requires modification when used for hollow or irregular forms of cross-section. It may be derived as follows: Consider the column of uniform cross-section shown by Fig. 1, and assume that it is slightly bent by the load, P; then the bending moment at the middle is

$$M = P\,d = p\,A\,d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where P = the load on the column,
d = the central deflection,
p = the load per unit area,
and A = the area of the cross-section.

From the theory of flexure,

$$\frac{M}{I} = \frac{p_1}{y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Where I = the moment of inertia of the cross-section about a diameter,
y = the distance of the outer fiber from the neutral axis,
and p_1 = the intensity of the outer fiber stress due to bending.

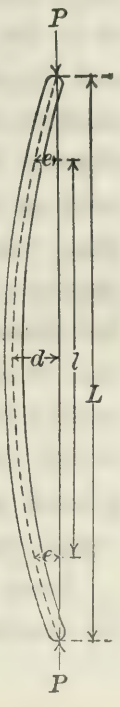


FIG. 1.

From Equations 1 and 2 the following relation obtains:

$$p\,A\,d = \frac{p_1\,I}{y}\,;\ \text{or}\ \frac{p_1}{p} = \frac{dy}{\rho^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

since $I = A\,\rho^2$, where ρ is the radius of gyration of the cross-section.

If the cross-section of the column is considered as being made up of two areas, as shown by Fig. 2, one of which, A_1, transmits the direct stress and the other, A_2, transmits the bending stress; then, if the outer

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fiber stress due to bending is equal to the direct stress, $P = p\ A$ may be put equal to $p_1\ A_1$; also, $I = A_2\rho^2$, giving

$$\frac{p_1}{p} = \frac{A_2}{A_1} = \frac{dy}{\rho^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

This relation obtains because any alteration of the breadth of a given cross-section does not alter the value of ρ. Adding unity to both sides of Equation 3 and putting $p + p_1 = f$, the strength to compression of the material, gives

$$p = \frac{f}{1 + \frac{dy}{\rho^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

In applying Equation 5 to columns and struts, the difficulty arises as to the determination of what function of the length applies to d, and it can only be approximated. The Rankine-Gordon formula assumes that the area, A_1, transmits only the direct stress and is not affected by the bending stress, and that the area, A_2, transmits the bending stress and is not affected by the direct stress; and further, that, when the column is about to break, the intensity of the direct stress on A_1 is then equal to the intensity of the bending stress on the outer fiber of A_2. This assumption gives the first approximation derived from theory, and errs on the side of safety, because the strength of the column, considered as a whole, is greater than that of the assumed compound column.

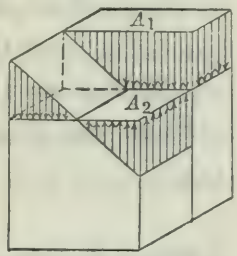

FIG. 2.

From the theory of flexure, the deflection of the column is given by the expression

$$d = \frac{f\,L^2}{\pi^2\,E\,y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

where E = Young's modulus of elasticity, L = the length of the column, and the other terms have the signification already given. This expression is the same as that obtained for a uniform beam supported at its extremities and carrying a load proportional to the ordinates of a sine curve; the deflection curve is then also a sine curve. Substituting for d in Equation 5 from Equation 6 gives

$$p = \frac{P}{A} = \frac{f}{1 + \frac{f\,L^2}{\pi^2\,E\,\rho^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

which is the form of the Rankine-Gordon formula for centrally loaded, round-ended columns. In order to modify it to include eccentrically loaded columns, assume that the sinusoidal deflection curve closely approximates to the parabola; then, from Fig. 1,

$$d = e + \frac{f\,l^2}{\pi^2\,E\,y^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

where e = the eccentricity of loading.

Substituting for d in Equation 5 from Equation 8 gives

$$p = \frac{P}{A} = \frac{f}{1 + \dfrac{e\,y}{\rho^2} + \dfrac{f\,l^2}{\pi^2\,E\,\rho^2}} \quad \ldots\ldots\ldots\ldots\ldots \quad (9)$$

which is the modified form of the Rankine-Gordon formula for eccentrically loaded columns.

The writer has given the further modification of the formula to include transversely loaded columns, which can be derived as follows: From Equations 9 and 4,

$$\frac{A_1}{A_2}\left(\frac{e\,y}{\rho^2} + \frac{f\,l^2}{\pi^2\,E\,\rho^2}\right) = 1$$

but

$$\frac{P\,e}{I} = \frac{f\,A_1\,e}{A_2\,\rho^2} = \frac{p_o}{y}$$

hence

$$\frac{p_o}{f} + \frac{f\,l^2}{\pi^2\,E\,\rho^2}\,\frac{A_1}{A_2} = 1$$

or

$$\frac{f}{f - p_o}\,\frac{f\,l^2}{\pi^2\,E\,\rho^2} = \frac{A_2}{A_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

giving, from Equation 4,

$$p = \frac{P}{A} = \frac{f}{1 + \dfrac{f\,l^2}{\pi^2\,E\,\rho^2}\,\dfrac{f}{f - p_o}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

To apply the formula given in Equation 10, a supplementary calculation is necessary in order to determine the load required to produce a given fiber stress, p_o. Writing Equation 10 in the form,

$$A_2 = \frac{A}{1 + \dfrac{\pi^2\,E\,\rho^2}{f\,l^2}\,\dfrac{f - p_o}{f}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

Now

$$\frac{p_o}{y} = \frac{M}{A_2\,\rho^2}, \text{ or } A_2 = \frac{M\,y}{p_o\,\rho^2},$$

and substituting for A_2 in Equation 12 gives

$$M = \frac{p_o \rho^2}{y} \frac{A}{1 + \frac{A}{\pi^2 E \rho^2} \frac{f - p_o}{f l^2}} \quad \ldots\ldots\ldots\ldots (13)$$

To determine M, the kind of normal loading must be specified; for a uniform load, $M = \frac{w l^2}{8}$, where w is the intensity of the load per unit length on the column; hence, for a given column, when M is given, p_o can be determined from Equation 13, and conversely.

For the uniform load, the shear stress, S, is $\frac{w l}{2}$, and this is taken up by the supports at the ends of the column; there is also the internal shear stress of the column to be considered. Usually, in a beam or girder, it is the shearing stresses which give rise to the bending moments; in columns, however, it is the variations of the bending moments which give rise to the shearing stresses. If, then, a column is bent to its proof deflection, so also should the web (if properly designed) come to its proof stress. Now consider the column as a beam supported at its extremities and bent to a sinusoidal curve. If w is the intensity of the load per unit area, and d is the central deflection, the bending moment

$$M = \frac{w d l^2}{\pi^2}, \text{ and } S = \frac{w d l}{\pi}$$

where S is the shearing force at one support. Hence,

$$S \frac{l}{\pi} = M = \frac{f I}{y} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Now $M = Pd$ for the column, and substituting for P and d from Equations 6 and 7 gives

$$S \frac{l}{\pi} = \frac{f A}{1 + \frac{f l^2}{\pi^2 E \rho^2}} \frac{f l^2}{\pi^2 E y}$$

$$\text{or } S = \frac{f I}{y} \frac{1}{\frac{l}{\pi} + \frac{\pi E \rho^2}{f l}} \quad \ldots\ldots\ldots\ldots\ldots (15)$$

From this equation the thickness of the web for box-columns can be determined; for braced columns, the area of the cross-section of the bracing is given by the equation, $a = \frac{S}{f} \sec. \theta$, where θ is the angle of the bracing with the direction of the shearing force. It follows,

then, from Equations 13 and 15, that in designing columns the external and internal shear stresses must be considered together.

The formulas given in Equations 7, 9, and 11 can be written in the generalized form

$$p = \frac{P}{A} = \frac{f}{1 + \frac{f}{f - p_o}\left(\frac{e\,y}{\rho^2} + \frac{f\,l^2}{\pi^2\,E\,\rho^2}\right)} \quad \ldots\ldots\ldots\ldots (16)$$

which may be regarded as the modified form of the Rankine-Gordon formula extended to include eccentric and transversely loaded columns. The formula only applies to solid columns, and requires modification when applied to columns of irregular or hollow forms of cross-section.

If a short, hollow, cylindrical tube be tested under direct compression in the testing machine, it fails by secondary flexure or wrinkling, and the tube breaks up into a series of waves. From a comparison of the results obtained from a large number of tests by the writer, it was shown that, as the tube becomes larger in diameter and the thickness less, the load producing failure becomes smaller, and it is not until the length of the tube is less than a wave length that the load producing failure approaches the resistance to compression of the material. Hence the true strength to compression of the tube is the load which produces the wave formation. From the experiments, and also from the analysis, it was found that the wave length varied as the square root of the area of the cross-section. This result leads to the following equation for the limiting load:

$$f_c = \frac{f}{1 + k\,\frac{\rho}{t}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

where f_c = the limiting load per unit of area on a column of one wave length,

f = the strength to compression of the material,

k = a constant,

ρ = the radius of gyration of the circular cross-section about a diameter,

t = the thickness of the circular cross-section.

Further experiments on columns of square, triangular, and H sections showed that the above formula applied, not only to circular sections, but to these sections as well, the coefficient, k, having a particular value for each cross-section.

Referring now to the Rankine-Gordon formula given in Equation 7: The value of f, the strength to compression, was assumed as constant, whereas the experiments show that it is a variable which involves the phenomena of secondary flexure, and therefore should be replaced by the value, f_c, given in Equation 17. The following formula is then obtained:

$$p = \frac{P}{A} = \frac{f}{1 + k\frac{\rho}{t} + \frac{f\,l^2}{\pi^2\,E\,\rho^2}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

This formula takes into consideration both primary and secondary flexure, and its solution involves the length and the figure and thickness of the cross-section of the column. Also, by giving values to k pertaining to the different figures of the cross-section, it can be applied generally to all forms of struts.

When eccentric and transverse loading require to be taken into consideration, the formula, by substitution from Equation 17 in Equation 16, becomes

$$p = \frac{P}{A} = \frac{f}{\left(1 + k\,\frac{\rho}{t}\right)\left(1 + \frac{e\,y}{\rho^2}\,\frac{f}{f - p_o}\right) + \frac{f\,l^2}{\pi^2\,E\,\rho^2}\,\frac{f}{f - p_o}} \qquad \ldots (19)$$

Table 1 gives the constants to be used in the foregoing formulas for different materials; these have been deduced from a critical examination of the tests on columns and from the writer's experiments. The value of the constant, c, is $\frac{f}{\pi^2\,E}$ for round-ended columns and $\frac{f}{4\,\pi^2\,E}$ for fixed-ended columns.

TABLE 1.

Material.	Strength to compression, f, in pounds per square inch.	Modulus of elasticity, E.	Round-ended columns, c.	Fixed-ended columns.
Nickel steel.....	120 000	31 000 000	$\frac{1}{2\,800}$	$\frac{1}{11\,200}$
Bessemer steel.....	110 000	31 000 000	$\frac{1}{2\,600}$	$\frac{1}{10\,400}$
Mild steel.....	80 000	30 000 000	$\frac{1}{4\,000}$	$\frac{1}{16\,000}$
Mild steel, annealed.....	60 000	30 000 000	$\frac{1}{5\,000}$	$\frac{1}{20\,000}$
Wrought iron, annealed...	55 000	26 000 000	$\frac{1}{4\,500}$	$\frac{1}{18\,000}$
Cast iron.....	110 000	14 000 000	$\frac{1}{1\,300}$	$\frac{1}{5\,200}$

The coefficient, k, is of the form $\frac{Nf}{E}$, where N represents the tabular number of the figure of the cross-section of the column. On Fig. 3 are shown the numbers obtained for a few types of cross-section; these have been obtained from tests on small specimens, for which reason they can only be considered as approximate.

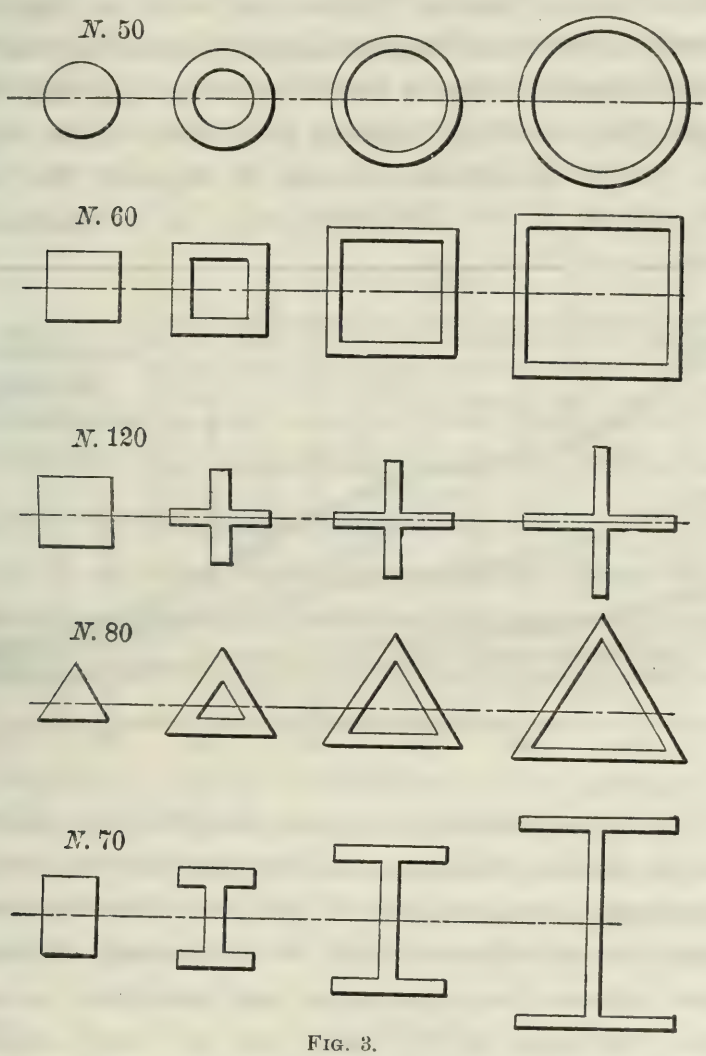

FIG. 3.

On Fig. 4 are shown the results of some tests by the writer on mild-steel columns, ½ in. in diameter. The values obtained for the solid, round-ended columns under central loading are shown by Curve *A*, and the plotted values from the Rankine-Gordon formula by Curve *2*. It will be noticed that these curves intersect at the value of $\frac{l}{\rho}$ = about 40 to 45. For values of $\frac{l}{\rho}$ less than 45 the values given by the formula

are slightly higher than the experimental ones, and for values of $\frac{l}{\rho}$ between 45 and 120 those given by the formula are somewhat less than those obtained from the experiments. For values of $\frac{l}{\rho}$ greater than 120 the calculated and experimental values sensibly agree with Curve *1*, obtained from Euler's formula. The writer has shown that the experimental curves can be closely approximated by assuming the deflection curve of the column to vary in some proportion less than the square of the length; the resulting formulas then become more complex and are of little use for the practical design of columns. For this reason the writer adheres to the Rankine-Gordon as a practical working

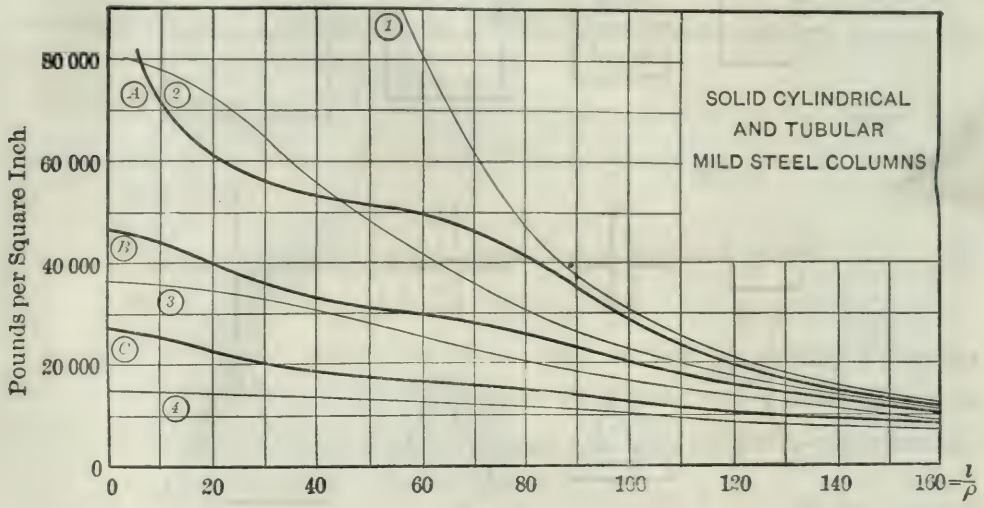

FIG. 4.

formula. The values obtained by its use, except for very short columns, always err on the side of safety, and the simplicity of its form must always recommend it to engineers. When evaluating the constants for the formula from experimental data, it is necessary to bear in mind the relations between the experimental and calculated curves, as just pointed out. The same remarks apply, but in a much smaller degree, when allowing for secondary flexure in the writer's formulas. Curves *B* and *3* show this. Curve *B* relates to a series of tests on centrally loaded, mild-steel tubes, $\frac{3}{4}$ in. in diameter and 18 S.W.G thick. These have been selected to show the influence of secondary flexure on the strength of such columns. Curve *3* shows the calculated values from the formula. Curves *C* and *4* relate to the same tubes but having an eccentricity of loading of $\frac{1}{4}$ in. In these experiments there is little

variation in the loads producing failure for a large range of the length, owing to the small deflection of the column under the applied load compared with the eccentricity of loading.

The values obtained by the use of the foregoing formulas will now be compared with the results obtained from some experimental tests on columns.*

The first results are those of compression tests on channel bars of latticed columns.† From the writer's experiments, the value of f, for solid, rolled, wrought-iron bars, is about 60 000 to 65 000 lb. per sq. in. Assuming the greater value, then Equation 17 becomes

$$f_c = \frac{65\,000}{1 + k\frac{\rho}{t}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (20)$$

From the sketches‡ of the different sections, it is only possible to approximate to the values of $\frac{\rho}{t}$ of the channel sections. Taking the value of ρ about an axis perpendicular to the long side of the section, and dividing by the thickness, gives $\frac{\rho}{t}$ approximately 9, 10, 11, and 12, for the 6, 8, 10, and 12-in. sections, respectively. These numbers give $k = 0.06$, approximately, and Equation 20 becomes

$$f_c = \frac{65\,000}{1 + 0.06\frac{\rho}{t}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

and the values of f_c for the 6, 8, 10, and 12-in. sections are 42 000, 40 500, 39 000, and 37 800 lb. per sq. in., respectively. On comparing these numbers with those given from the tests on the shorter lengths of the channel bars they will be found to be in close agreement.

Substituting from Equation 21 in the Rankine-Gordon formula gives

$$p = \frac{P}{A} = \frac{65\,000}{1 + 0.06\frac{\rho}{t} + \frac{1}{4\,300}\frac{l^2}{\rho^2}}$$

E being taken at 28 000 000. The data in the sketches and tables are not sufficiently precise to allow of any great accuracy in determining

*Some of the tables showing the results of these tests are contained in Professor Lanza's "Applied Mechanics" (Ninth edition), and some are in the *Transactions* of this Society.

†Lanza, "Applied Mechanics," pp. 422–423.

‡Lanza, "Applied Mechanics," pp. 427–428.

the values of ρ, of which two are to be considered, one in a plane parallel to the lacing and the other in a plane perpendicular thereto. Using the smaller values in the formula gives numerical results which approximate to the experimental values in the tables.* The constant, $\frac{1}{4\,300}$, has been deduced for round-ended columns. For the tests given in the tables the friction of the pins prevents this condition from being satisfied, and therefore the constant requires to be modified for these tests; a value of about $\frac{1}{5\,000}$ should be used.

The manner of failure, as stated in the tables, gives but little information as to the way in which the lacing was affected when failure of the column took place. As already pointed out, in a laced column the proportions of the lacing depend on the shear stresses due to primary flexure of the column. Reverting to Equation 15, and applying it to the test column, No. 463,† gives the thrust in the lacing as about 1 750 lb. For this column, the following are the approximate data: From Equation 21, $f = 42\,000$ lb. per sq. in., $A = 4.68$, $\rho = 3$ in. (nearly), $l = 240$ in., and $y = 4.5$. Substituting these values in Equation 15 gives

$$S = \frac{42\,000 \times 4.68}{4.5} \times \frac{3}{\frac{80}{\pi} + \frac{670\,\pi}{80}} = 2\,500 \text{ lb.}$$

The lacing slopes at 45° (nearly), giving

$$P = S \sec. \theta = 2\,500 \times \sqrt{2} = 3\,500 \text{ lb. (nearly).}$$

This thrust is taken up on two bars of the lacing, giving 1 750 lb. on each.

The size adopted for the lacing was 1.95 by $\frac{1}{4}$ in. in cross-section and 11.3 in. long between rivet centers. Assuming the bar to have been hinged at the ends, the safe thrust it would carry, by Equation 7, would be

$$P = \frac{65\,000 \times 1.95 \times \frac{1}{4}}{1 + \frac{24\,500}{4\,300}} = 4\,850 \text{ lb., approximately.}$$

As the calculated thrust was only 1 750 lb., the bracing was of ample strength. This seems to have been the case with nearly all the experi-

* Lanza, "Applied Mechanics," pp. 422, 423, 425.

† Lanza, "Applied Mechanics," p. 425.

ments given in these tables,* more especially as no particular failures of the lacing were recorded.

In addition to the shear stress due to primary flexure, there also arises the question of the proportions to be adopted for the lacing when secondary flexure is considered. If the lacing is made uniform throughout the column, and is of sufficient strength to transmit the shear stresses, it is also of ample strength to resist deformation due to secondary flexure. The writer has shown that, in the case of the usual box- and H-sections, the wave length of the wave formation is less than the breadth or width of the section. In a laced column, the channel bars are forced by the lacing to take up a wave length of twice the length of the lattice spacing, and this quantity is always greater than the size of the section. Under these conditions the stresses set up in the lacing due to secondary flexure are small compared with the shear stress at the ends of the column.

In a few tests of Phœnix columns,† the constants deduced therefrom, on substitution in Equation 18, give

$$p = \frac{P}{A} = \frac{70\,000}{1 + 0.04\,\frac{\rho}{t} + \frac{1}{4\,000}\,\frac{l^2}{\rho^2}}.$$

In using the formula, it is to be noted that the columns were tested with flat ends, and therefore the ends of the column were sensibly fixed. For these columns, the constant, $\frac{1}{4\,000}$, is multiplied by $\frac{1}{4}$. The coefficient, $k = 0.04$, for these sections, has the smallest value yet obtained from tests on columns. Therefore, they give the column of maximum strength for a given quantity of material. The writer has previously drawn attention to the advantage of stiffened and cellular forms of cross-sections for columns.

The results of a number of tests on wrought-iron pipe columns at the Massachusetts Institute of Technology‡ will be considered next. The deduced value of the constants, on substitution in Equation 18, gives

$$p = \frac{P}{A} = \frac{65\,000}{1 + \frac{1}{10}\,\frac{\rho}{t} + \frac{1}{4\,000}\,\frac{l^2}{\rho^2}}.$$

* Lanza, "Applied Mechanics," pp. 422, 423, 425.
† Lanza, "Applied Mechanics," p. 425.
‡ Lanza, "Applied Mechanics," p. 440.

This formula agrees closely with the writer's experiments, on tubes of much smaller diameters, in the laboratory of Trinity College, Dublin.

Now examine some tests on wrought-iron Z-bar columns by C. L. Strobel, M. Am. Soc. C. E.* The Z-irons used in making the columns were $2\frac{1}{2}$ by 3 by $2\frac{1}{2}$-in. and $\frac{5}{16}$ in. thick. The constants deduced from these tests, on substitution in Equation 18, give

$$p = \frac{P}{A} = \frac{65\,000}{1 + 0.4 + \frac{1}{4\,000}\frac{l^2}{\rho^2}}.$$

The columns were tested with fixed ends, and therefore the constant, $\frac{1}{4\,000}$, is to be multiplied by $\frac{1}{4}$. The value of $k\frac{\rho}{t}$, for the section of these columns, cannot be given until further experimental data have been obtained.

In some tests on eight full-sized Bessemer steel bridge columns by the late J. G. Dagron, M. Am. Soc. C. E.,† the deduced value of the constants, on substitution in Equation 18, gives

$$p = \frac{P}{A} = \frac{80\,000}{1 + 0.5 + \frac{1}{4\,000}\frac{l^2}{\rho^2}}.$$

These were latticed columns, and were tested with pin ends. It is necessary to make a slight allowance for the friction of the pins, which increases the value of the constant from $\frac{1}{4\,000}$ to about $\frac{1}{6\,000}$. The lattice bars were $1\frac{3}{4}$ by $\frac{1}{4}$ by $14\frac{1}{2}$ in., center spaced, and 18 in. apart. Reverting to Equation 15, and substituting the data from the table, the thrust in the lattice bars at the ends is approximately 3 500 lb., and the breaking load on the lattice bars, calculated from Equation 7, gives 3 300 lb., showing that the proportions for the bracing were closely correct. The columns invariably failed by crippling, the lattice bars at the ends of the columns remaining intact.

The data, details, and results of some tests on mild-steel and wrought-iron columns, by Professors Talbot and Moore‡ will now be

* *Transactions*, Am. Soc. C. E., Vol. XVIII, 1888, p. 103; also Lanza, "Applied Mechanics," p. 439.

† *Transactions*, Am. Soc. C. E., Vol. XX, 1889, p. 254; also Lanza, "Applied Mechanics," p. 495.

‡ *Transactions*, Am. Soc. C. E., Vol. LXV, 1909, p. 202; also, *Bulletin No. 44*. University of Illinois, June, 1910.

examined. The deduced value of the constants from Test 17 on the mild-steel column, No. 1, on substitution in Equation 18, gives

$$p = \frac{P}{A} = \frac{60\,000}{1 + 1.4 + \frac{1}{4\,600}\frac{l^2}{\rho^2}}.$$

The tension tests show that the steel was of a very mild quality, and for this reason the value of $f = 60\,000$ has been adopted. The value of the coefficient, k, is approximately 0.09. The column failed by the sudden buckling of the flange between adjacent lattice bars, and therefore the ends of the column may be regarded as having been partly fixed. Reverting to Equation 15, and substituting the values from the data the shear stress is determined as being approximately 3 800 lb., and the thrust in two lattice bars about 4 250 lb. The lattice bars were $1\frac{1}{4}$ by $\frac{7}{16}$ by 18 in., and the breaking load on these bars, calculated by Equation 5, gives 6 500 lb., and for two bars 13 000 lb., showing that they were of ample strength. It will be noted that the column failed by crippling. In Test No. 1* the lattice bars were 1 by $\frac{1}{4}$ in. The breaking load on these bars, calculated by Equation 7, gives 1 000 lb. (nearly), and 2 000 lb. for two bars, showing that they had only half the required strength. In this test the column failed by the buckling of the lacing.

The deduced value of the constants, from the tests on wrought-iron columns, on substitution in Equation 18, gives

$$p = \frac{P}{A} = \frac{55\,000}{1 + 0.6 + \frac{1}{5\,000}\frac{l^2}{\rho^2}}$$

and the value of the coefficient, k, approximately 0.08. These columns failed by crippling, no failure of the lacing being recorded. Calculating the value of the shear stress from Equation 15 shows that the lacing was of ample strength.

The object of this paper is to draw attention to the merits of the Rankine-Gordon formula, and of the writer's formula for co-ordinating the experimental data obtained from tests on columns. The formula takes into consideration eccentric and normal loading and secondary flexure, which covers most of the factors that arise in considering problems on columns; moreover, the formula is derived in a logical way, which theory shows to be a first and true approximation of the problem,

* *Bulletin No. 44*, University of Illinois, Table 7, p. 35.

a merit which, as far as the writer is aware, is not possessed by any other column formula. From the standpoint of the engineer, the use of theory, for the purposes of design, is to act as a guide, and it is essential to know within what limits it may be safely relied upon. In this respect, the derivation of the formula, as already pointed out, errs on the side of safety, and this is also true when applying it to evaluate the shear stresses set up in the column. Other formulas in use for the design of columns, such as the straight-line formula, can be modified to allow for secondary flexure. For instance, the straight-line formula can be put in the form,

$$p = f - a\frac{\rho}{t} - b\frac{l}{\rho}$$

where a and b are constants.

This formula, although useful within a limited range of $\frac{l}{\rho}$, for purposes of rapid design, can only be roughly modified to include eccentric and transverse loading; and, as a formula for co-ordinating the results of experimental tests, is of little value.

The writer does not claim that any great accuracy has been obtained in deducing the constants from the experimental data in the paper. The data given with the tests are not sufficient, and the range of the experiments is too limited to allow of more than first approximations being obtained. To dissect and co-ordinate the mass of experimental data now in existence is beyond the powers of any one man, and the work can only be carried through by a body such as the American Society of Civil Engineers, who, the writer understands, has formed a Special Committee to deal with the subject. A co-ordination of the experimental data on column tests would be of great value to the Engineering Profession, and the American Society of Civil Engineers is to be congratulated on being the first to take the matter in hand.

APPENDIX.

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DISCUSSION

Mr. Godfrey.

EDWARD GODFREY, M. AM. SOC. C. E. (by letter).—This paper is timely and important, not so much because it adds complexity to the already overburdened subject of the theoretical strength of columns, but because it affords an opportunity for a skeptical examination of the basis of all these column formulas, and particularly because any adverse criticism of the basic formulas will no doubt find a worthy champion in the able author.

In the technical press, the writer has repeatedly assailed both the Gordon-Rankine and the Euler formulas for columns. A paper on this subject, which if not denied ought to have revolutionized the subject, he had difficulty in finding a publisher to accept. It was not controverted when published, and it has not revolutionized the subject; the writer expected nothing of the sort; "what ought to be" and "what is" are separate and distinct things. It takes many years to pry accepted standards loose from a body of professional men, even though these standards are clearly proven false. In the meantime the writer has observed and demonstrated in his practice and reading that confidence in the Euler and Gordon-Rankine formulas has resulted in failure, as the error is so great.

When the writer was a student he swallowed the arguments of his textbooks largely because of the authority behind them. Since he has "put away childish things" he appreciates the fact that the highest authorities may err, and that error may be in the very subject that they know best. He accepted the apparent logic of the derivation of these formulas in those days, just as he would now probably accept what the textbooks state regarding the supposed strength of presumably reinforced concrete columns, which are constantly proving, by failing under a fraction of that strength, that such textbooks are wrong. Now, in reviewing his textbooks, he fails to discover any logical basis for either the Euler or the Gordon-Rankine formulas. If there is such basis, it is not stated in these books.

Textbooks say of the Euler load that "under this load the column just begins to deflect, and will under a constant load retain any deflection which may be given to it, within the elastic limit of the material." The writer can find no logical proof of this in the derivation of the Euler formula in these same books. It happens that the Euler load is one that will double any initial bow in a column. If an end load adds 100% to the initial bow of a round-ended column (more properly, one with knife-edge bearings), it will, by reason of this added bow, add another like amount because of that added deflection, and again another and another, and so on *ad infinitum,* or until the column fails. So that, whatever initial bow the column has to start

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with, it will fail at the Euler load. A column may be perfectly straight, that is, have an initial bow of infinitesimal amount: it will fail at the Euler load, and will not carry any more load than at its first measurable deflection. A similar column may have an initial bow of, say, $\frac{1}{8}$ in.; it will continue to deflect, but will not fail until the load is twice that which gives a total deflection of $\frac{1}{4}$ in. Both columns will sustain the same ultimate load (assuming that they are slender columns), though the originally imperfect one will deflect much more before reaching its ultimate capacity. These facts can be proven by the theory of flexure. They show one of the anomalies of the theory of columns. The writer believes that one would have to search through engineering literature a long time before he would find any statement of these facts, and yet they have a tremendously important bearing on the subject of the strength of columns.

The Euler load is independent of the tensile or compressive strength of the steel, depending only on the modulus of elasticity, which is practically the same for all grades of steel. The Euler load is the absolute maximum that any column can take, no matter how high the elastic limit or the ultimate strength of the steel may be, and two slender columns, of the hardest and of the softest steel, respectively, will have practically the same ultimate strength. These are also anomalies, and are very difficult to find in engineering literature. They also have an important bearing on the subject of the strength of columns.

While the Euler load is the greatest that any column could take, it has practical application only to slender columns. Short columns, by reason of the limiting compressive strength of the metal, cannot sustain loads approaching the Euler load, but will fail by crushing or buckling. Hence some other formula must be used for shorter columns. None can be correct, however, that shows greater ultimate strength for any column than that shown by the Euler formula, and herein is where the Gordon-Rankine formula is in error, at least, in its application in American books. Rankine does not point out this limitation in his derivation of the formula.* His statement: "The greatest deflection [of a rectangular column] consistent with safety is directly as the square of the length, and inversely as the thickness," is not sufficiently full. The deflection which counts is not the initial bow, which might be conceded to be constant for similar columns, but the resulting bow after the load is applied and equilibrium is established. There is no relation between this deflection and the dimensions of the column, for it is a function of the load itself. Any treatment that fails to recognize this is incomplete and is likely to result in error.

* "Applied Mechanics," p. 361.

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Dr. Lilly, in effect, ties up his Gordon-Rankine formula with the Euler formula when he recognizes that the deflection or curvature in the column will limit its carrying capacity. His values of p cannot exceed the Euler unit stress. The constant of his Gordon-Rankine formula is thus deduced from purely theoretical reasoning. This is eminently better than empirical determination of the constant as the latter has worked out.

A common value for the constant of the Gordon-Rankine formula for round-ended columns is $\frac{1}{18\,000}$. This, with a value of 50 000 for f, gives, for the ultimate strength of a column having a ratio of slenderness of 240, a unit stress of 11 910 lb. per sq. in. (Handbooks work this out for the busy user.) The Euler load for this column is only 5 140 lb. per sq. in. This is the absolute maximum load that any column could take, and yet a formula in general use appears to show that it can take 132% more than this. Here is the count which the writer would urge against the Gordon-Rankine formula, and he has known failure to result from confidence in this same formula with the constants commonly used.

The writer believes that the Gordon-Rankine formula fails to meet the needs of the practical design of columns, and that a straight-line formula is far superior. It gives results closer to those obtained from experiments, and there are several reasons why it should.

Columns, as commercially manufactured, are imperfect, of necessity, and a formula for their design should take into account this fact. They are not in true alignment, and their end connections are not always central. The writer has shown* that, if proportionate imperfections are assumed in columns, a purely theoretical formula can be deduced, which, though very complex, gives a locus which is almost straight for a large part of its length and agrees closely with the commonly used straight-line column formulas.

The ultimate strengths of test columns fall away rapidly after the range of very short columns is passed. This is probably because of local crimping or buckling of the metal, but it is a fact which must be dealt with in the treatment of columns. The Gordon-Rankine curve does not take this shape, hence it fails to meet this condition. The straight-line formula does meet this condition, for the locus falls away from the start.

The straight-line formula has the further advantage that it discourages the use of slender columns. Slender compression members may be weak by reason of their own weight, or, if in a vertical position, an accidental blow may cause them to fail.

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* *Railway Age Gazette*, July 2d, 1909.

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In the writer's opinion, the whole subject of columns in engineering textbooks should be re-written, and its theoretical treatment simplified, instead of being rendered more complex. A large part of the engineering literature on this subject could be expunged with resulting benefit.

It is manifestly impossible to evolve a formula which will show close agreement with any comprehensive series of tests, for the reason that similar columns show discordant results. The exact strength of structural steel columns cannot be predicted, because imperfections of manufacture enter so largely in the results. Approximate results are all that can be expected, and simple theory answers this purpose just as well as the most complex theory ever devised.

In this re-casting of column literature, the importance of the Euler load should be emphasized, not as a load which the column can hold in equilibrium, conveying the idea that there is surplus strength in the column, but as the extreme limit of its carrying capacity.

Another fact of great importance which should be emphasized is that slender columns of all grades of steel are of practically equal strength. Working formulas should recognize this, and values should converge for long columns in low and high steels. Nickel steel struts of light dimensions are not economical, because their strength is practically the same as for soft steel, though they cost much more.

The converging of the strengths of columns of different grades of steel as the lengths increase has been illustrated by some tests* made by J. A. L. Waddell, M. Am. Soc. C. E. With similar columns of carbon steel and nickel steel in which $\frac{l}{r}$ was 27, the average strength of the latter was 75% greater than that of the former; with others, in which $\frac{l}{r}$ was 81, the nickel steel columns averaged only 47% stronger. This indicates clearly the convergence to equality that theory proves must exist in slender columns of high and low steel.

On the Continent of Europe the Euler formula seems to be the standard for the design of columns. This is a grave error which American engineers do not commit. The Euler formula has no application whatever to columns of ordinary lengths, as used in bridges and buildings, for the values increase as short lengths are approached, and it would require steel of almost unlimited strength to satisfy the formula and hold up under the compression.

A short time ago, a gas-holder post, in a structure in Germany, failed, with disastrous results. The column was designed by the Euler formula, which was one of the errors made by the designers,

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 250.

for it was not (as considered by them) a slender column. The gravest error made in the design was the use of batten-plates instead of lattice. Another woful lack in the theoretical treatment of columns is that of emphasis on the extreme importance of some means of carrying shear in both rectangular directions in the column. Batten-plates will not do this in any adequate degree. This column was made up of two 5-in. channels held together by a few pairs of small tie-plates. As the writer has pointed out,* these tie-plates, or batten-plates, cannot prevent the channels from bowing and acting practically as separate slender columns. Fig. 5 shows this column and how and why it could fail as a slender column. It is surprising that a high European authority, instead of condemning this flimsy construction, on the basis of common sense and theory, delivered the following: Mr. Godfrey.

"The use of tie-plated columns, when the section is assumed to be integral, may lead to constructions which do not afford adequate security under loading of unusual character."

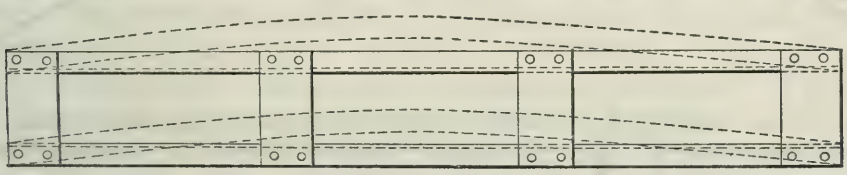

FIG. 5.

Much is said of the impracticability of securing true hinged or pin ends on columns in testing them, and the idea is prevalent that most compression members are in fact practically fixed-ended in structures. This is more of the misinformation of engineering literature. It is much easier to get a practically pin-ended member in a structure than in a testing machine. In the latter, the rigid heads and friction on the pins hold the member under test almost rigid at the ends. In a bridge, the compression members are only insecurely held by other members as weak as, or weaker than, themselves.

A case which came under the writer's notice is that of the strut of a jib crane shown in Fig. 6. The designer considered this as fixed-ended, and proportioned it by the Gordon-Rankine formula. It is no wonder that failure occurred. The writer considered the strut as of the slenderness of one of the channels, and pin-ended. This is the only reasonable way to treat it. It could fail by the bowing of the two channels, as indicated. The single pair of batten-plates could offer but little resistance. The gusset-plates, to which the ends are attached, are more nearly like ideal pin-ended connections than pins would be, for there is practically no resistance against rotation.

* *Engineering News*, July 27th, and September 28th, 1911.

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These are some of the things that could be written into the subject of columns to replace a vast amount of meaningless mathematics.

The subject of reinforced concrete columns has sprung up with a rank growth of mathematical nonsense which every great reinforced concrete disaster disproves. Tests are interpreted as applying to construction, while they do not embody the essential features necessary to safe construction. Columns utterly lacking in toughness are tested with infinite care (in testing) in order to preserve their evanescent strength; then such columns are built into a structure where toughness is an essential characteristic. Is it any wonder that the Engineering Profession is degraded by periodic wrecks, when its leaders show such lack of common sense?

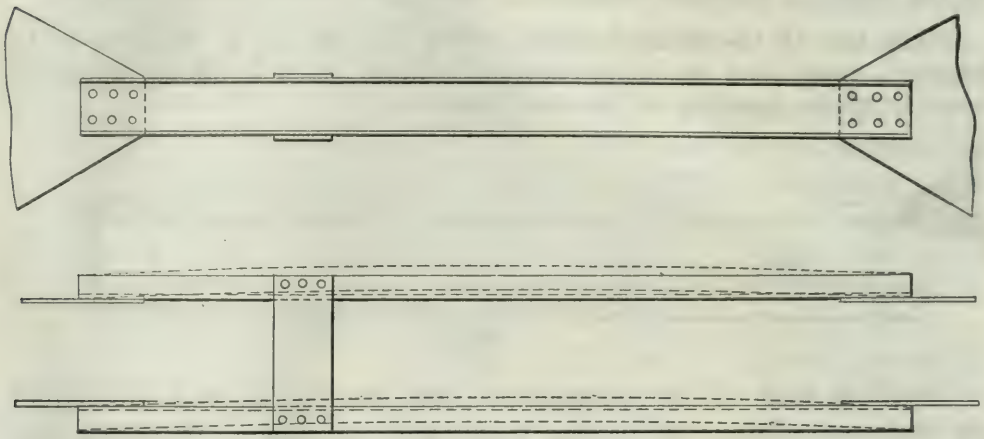

FIG. 6.

Mr. Eckersley.

J. O. ECKERSLEY, M. AM. SOC. C. E. (by letter).—This brief discussion of Mr. Lilly's suggestive paper was prompted by the question: Would it not repay the endeavor to derive a more rigorous treatment for the strength of columns? Science is truly a function of the epoch in its ideals and achievements, and technical knowledge, as we understand it to-day, is becoming pre-eminently mathematical, and justly so, for mathematics unchallenged governs the appreciation and mastery of the physical sciences in theory and practice.

Even though one has the feeling that the solution, in an absolute sense, founded on abstract mathematical reasoning, will be found impossible; at the same time, one recognizes the importance of the endeavor to attain precisely this result.

Although the demands made on mathematical science, to follow Mr. Lilly's paper, are not great, yet, if the rough and ready reasoning of the engineer gave way to the more rigorous analysis, a considerable knowledge in the realm of higher mathematics would be required.

In general, engineers, almost unanimously, regard analyses involving higher mathematics as dispensable. Judging from past experience, however, there appears to be every inducement for the mathematical

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engineer to continue his studies with unflagging zeal and confident optimism, for there is little reason to doubt, and many reasons to expect, greater achievements through just such persistent pursuit of scientific investigation.

This attitude of engineers toward analyses is clearly shown in their endeavor to simplify everything, an endeavor economically and entirely commendable, which, however, introduces simplifying assumptions such that it is questionable whether we may with propriety consider them valid, at least from a mathematical viewpoint.

The syncopation, we might say, of the column formula is no exception. For instance, Rankine's, written in the form, $y = a\ (1 + bx^2)^{-1}$, is a cubic equation; this was simplified to a quadratic by substituting a parabola in the form of $y = c - dx^2$; and, in due time, it was further reduced to the linear form, $y = e - fx$, where the letters, a to f, inclusive, represent empirical constants determined in every case as if the simplifying assumptions of the theory of flexure, initial straightness, homogeneity, central loading, etc., were rigorously obtained. The questioned validity of the deductions from such formulas is shown by the fact that the results of experiment and the deductions are not confirmatory. More exact mathematical treatment of the question than that given by the above formulas has been advanced by several writers, English, French, German, and American; also, the effects of eccentricity and local bending between points of attachment of lattice bars are not new. Mr. Lilly's treatment of the subject, in common with others, has introduced approximations at divers places in the analysis, which permit of criticism from the mathematical standpoint.

To compare some of the salient points of difference between what we may term the rigorous, as compared with the usual, approximate treatment, let us consider the following parallel columns for reference:

Rigorous.	Usual Approximation.
1st. When a column is subjected to strain, its dimensions in every direction are altered, the lateral changes following a law similar to that of Hooke.	1st. Changes of dimensions are construed as negligible.
2d. Strains do occur in all the three mutually perpendicular planes of reference.	2d. The strains are assumed to be entirely co-planar, *i. e.*, in one plane.
3d. A torque would, in general, exist.	3d. Irrotational stress is assumed.
4th. The elastic curve, when the column weight is considered, is not a symmetrical curve.	4th. The weight of the column is neglected, which leads to a symmetrical elastic curve.

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Rigorous.	Usual Approximation.
5th. With suitable lateral forces, the so-called neutral plane may be broken or even discontinuous.	5th. So far as the geometry of the neutral line is concerned, it is regarded as a continuous curved line having a tangent and osculating plane.
6th. The curvature would, in general, be tortuous.	6th. In the curvature formula, it is assumed that $dl = dx$, which reduces the exact expression, $\frac{dx d^2 y}{dl^3}$, to $\frac{d^2 y}{dx^2}$. (Where the curve is such that a great difference in x occurs for a small difference in y, the exact form should be retained.)
7th. The section after strain is actually curved.	7th. The section is assumed to remain plane.
8th. Vertical shear-deformation exists.	8th. Vertical shear-deformation is neglected.
9th. The conjugate effect of transverse strain really transforms the surface of a rectangular sectioned column into an anticlastic surface.	9th. Conjugate effect of transverse strain is neglected.

To examine the column by the more exact treatment, recourse must be had to the general equations of the equilibrium of an elastic solid, and would lead to expressions involving determinants, elliptic functions, Bessel equations, and what not, and is beyond the scope of this discussion. The column would be introduced by definition, endowed with certain ideal properties, and subjected to certain conditions not confined to empiric verification, just as a set of necessary and sufficient axioms is adopted for the various geometries, and the translation, into the language of the applied mathematical treatment, would be accomplished by substituting as many of the properties of the materials as could be experimentally determined or inferred, in lieu of the ideal.

The essential difference between such an analysis and the usual method is that evanescent quantities of a higher order would be retained and their final elimination made when the derivative governing the desired degree of accuracy was determined; while the usual method neglects certain factors in the initial assumptions and later approximations, believing that their effect on the final result is negligible,

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which, of course, is admissible when the degree of approximation permits, that is, when the displacements are expressed as functions of some selected system of spatial co-ordinates and are developed into series—the approximation ends with the first derivative, and we have the usual Hooke's assumption, of a plane remaining plane under strain. This order of accuracy, however, is only an approximation; yet, under certain conditions, it is sufficiently close.

What the writer wishes to bring out clearly is this: A vast literature has developed on the subject, both at home and abroad, and nowhere has he found what could be called a rigorous solution from the purely "mathematical theory of elasticity" point of view. The question in mind while writing this is: Would it not be profitable to attack the problem from such a point of view, and then use this as a means toward the interpretation and co-ordination of the phenomena? What if the analysis does involve the theory of probabilities, determinants, and higher functions? It is the pursuit along these lines which offers any reasonable excuse to engineers for the existence of anything other than a straight-line formula; for the problem from an engineer's point of view is simply this: Primarily, for very short prisms (so short that no lateral forces act), the crushing strength represents the value of the function. Secondarily, for very long prisms, in general (even though lateral forces act), the bending value is the controlling factor, and represents the function. Hence, all that the engineer desires is a rational formula giving a continuous relation between these two limiting values, even though it contains approximations.

It is obvious how complex the relations between the crushing, shearing, and bending values of the material would be for intermediate lengths, aside from the influence of local bending due to lattice bars, etc. The true form of the expression involving these would be of little interest save to the mathematical engineer, for it must be conceded that, from a practical point of view, a straight-line formula can be used successfully in the design of columns.

Finally, Mr. Lilly is to be congratulated and encouraged to continue his investigations, which will, it is believed, approach more nearly the more rigorous treatment as he considers the neglected factors.

Mr. Prichard.

Henry S. Prichard, M. Am. Soc. C. E. (by letter).—The author considers the Rankine-Gordon formula to be the best of the various formulas which have been put forward to determine the strength of columns, and in support of his opinion he quotes the statement that: "The theoretic basis of Rankine's formula seems far more satisfactory than that of any other which has been proposed"; but he qualifies his opinion in respect to "theoretic basis," by stating that an assumption is made in the derivation of the formula which "errs on the side of safety."

Mr Prichard.

In dealing with the practical side of his subject, the author gives, in Fig. 4, a curve, *A*, showing the results of some tests which he made on mild-steel solid columns, ½ in. in diameter, and another curve showing the results obtained by using Rankine's formula with constants, which he, the author, has deduced for mild steel; and he states that he "has shown that the experimental curves can be closely approximated by assuming the deflection curve of the column to vary in some proportion less than the square of the length"; that "the resulting formulas then become more complex and are of little use for the practical design of columns," and that "for this reason" he "adheres to the Rankine-Gordon as a practical working formula."

In this comparison of the Rankine-Gordon formula with the results of experiments, the constant used as "the strength to compression" is 80 000 lb. per sq. in. The writer does not share the author's opinions as to the theoretical and practical excellence of the Rankine formula, and he does not approve of the use of such a high unit stress for "the strength to compression."

Rankine, before giving his analysis of beams and columns, states that "the elasticity of every solid is sensibly perfect when the strain does not exceed a certain limit," that "Hooke's Law—*'ut tensio sic vis'*—is sensibly true for all relations between strains and stresses," and that "this condition is fulfilled in nearly all cases in which the stresses are within the limit of proof strength"; and he bases his analyses on these fundamental statements; in other words, he predicates ideal conditions.

In his analysis of beams, he derives equations for deflections based on the fact—quite true for beams—that the entire stress in the extreme fiber is due to bending. Subsequently, he makes the mistake of applying one of these equations to the deflection of columns, entirely overlooking or neglecting the fact that, in all cases of ordinary columns, part, at least, of the stress in the extreme fiber is due to direct compression. This mistake leads to an erroneous formula for columns, which, on the strength of Rankine's endorsement, has deceived and has continued to deceive engineers, and interfere with a correct understanding of the subject, for more than fifty years.

The error is reflected in the author's Equation 6 for deflection of columns, which he makes a factor of the total stress intensity in the extreme fiber, f, instead of a factor of $f - p$, as it should be (p being the load per unit area). The author prefaces this substitution with an unwarranted assumption, which, as he states, "errs," and gives what he terms "the first approximation." When f has a value of 80 000 lb. per sq. in. and the length is 60.8 times the radius of gyration, the effect of this substitution is to change the result of his Equation 7 from 80 000 to 40 000 lb. per sq. in.

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It is astonishing that Rankine should have made, or acquiesced in, this error in regard to the deflection of columns, as a centrally loaded column, made of perfectly elastic material and having an initially straight axis, is truly a spring, and he has proved the following:*

"THEOREM. *That a spring of a given length and section, to the ends of whose neutral surface a pair of forces are applied, will not be bent if those forces are less than a certain finite magnitude.*"

He develops the equation, $P = \frac{\pi^2 E I}{l^2}$ (which is Euler's), and adds, "and this *finite quantity* is the *smallest force which will bend the given spring* in the manner proposed." For instance, a centrally loaded, perfectly elastic, initially straight spring, with a modulus of elasticity of 30 000 000 and a ratio of length to radius of gyration of 60.8, will not be bent if the load is less than 80 000 lb. per sq. in.

There is another limit to the value of P; it cannot, of course, be greater than $f A$ (limiting stress intensity times cross-sectional area), and this latter limit governs for short columns, as the force required to strain such columns to the limiting stress intensity is less than that which it would take to bend them, if they were not thus limited. For instance, if a steel spring be conceived as having a ratio of length to radius of gyration of less than 60.8, and if the limiting stress intensity is 80 000 lb. per sq. in., the stress will reach this limit under a load of 80 000 lb. per sq. in. without any preliminary bending.

In view of the prevalent uncertainty as to the theory of perfectly elastic, initially straight, centrally loaded columns, the writer suggests that the author state whether he accepts this theorem of Rankine's; and if he does not accept it, that he indicate wherein he considers Rankine's "proof" to be in error.

For the limiting stress intensity given by the author (80 000 lb. per sq. in.), the limiting line, on Fig. 4 (beyond which initially straight, perfectly elastic, mild-steel columns will either crush or become plastic and flow, if short, or begin to bend and almost immediately fail,† if long), will follow the 80 000-lb. per sq. in. line from $\frac{l}{\rho} = 0$ to its intersection with Euler's curve at $\frac{l}{\rho} = 60.8$, and from

*"Applied Mechanics," Eleventh edition, pp. 351–352.

† In a paper on the "Theory of the Ideal Column," in *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), pp. 100–102, William Cain, M. Am. Soc. C. E., has shown: "first, that Euler's formula gives the load at which bending just begins, and, second, that a very small increase to this load insures failure." As a numerical illustration, a column pivoted at the ends. 325 in. long, was assumed as built up of two 5-in. channels. The inch being the unit, $A = 3.9$, $I = 14.8$, and $E = 29\,000\,000$ lb. per sq. in. Euler's formula gives the load that causes incipient bending as 40 105 lb. It was computed by an exact formula that an increase of the load of only 5 lb. caused a deflection of 3.44 in. at the center, with a resultant stress on the most compressed fiber greater than the elastic limit. Finally, an increase of 2 or 3 lb. more would entail rupture, or a breaking in two of the column.

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this point it will follow Euler's curve for all longer columns. The difference between this theoretically correct line and the line, Curve *2*, of the author's adaptation of Rankine's formula, shows the latter to be in error as a theoretical formula, for such ideal steel under such ideal conditions, from 0 to 50 per cent.

Of course, steel which crushes at 80 000 lb. per sq. in. is not perfectly elastic to this limit, and, of course, the other ideal assumptions of theory are not fully realized. Actually, columns are never quite initially straight and centrally loaded. There is always some initial deviation between the axis and the line of thrust (though it is not always appreciable), and this causes some deflection under any load, though in many cases it is very slight and in some it is not appreciable until the load is nearly equal to either Euler's limit, if the column is long, or to the yield point, if it is of moderate length.* Actually, the limiting stress at which failure occurs, in wrought-iron and steel columns which are too short to have Euler's formula apply, is not constant (even for material of the same quality), as indicated by theory, but varies from a stress somewhat below the yield point, for the longest columns of this class, to the yield point, for short columns, and to the crushing stress for very short ones. This is due to the great reduction in stiffness which takes place at the yield point and to the partial reduction in stiffness from imperfections in elasticity, which develops shortly before the yield point is reached.

The loss in stiffness necessary to cause failure by bending grows less as length increases; hence, the load required to cause the failure of columns of this class grows less as length increases, even when great care is taken to reduce to a minimum the initial deviation between the axis and the line of thrust.

The fact that the strength of wrought-iron and soft or medium steel columns of moderate length is limited to about the yield point, is well illustrated in Tables 2 and 3, and many other series of column tests could be cited if necessary. In fact, the writer does not know of any such columns of moderate length which sustained loads greater than the yield point of the material. In experiments, short columns, solid or of thick material, do, temporarily, at least, sustain loads in excess of the yield point, and their ability to do so has a value as a

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* In illustration, the following tests are cited:

A 4-in., wrought-iron, I-beam column, 6 ft. 6¼ in. long with rounded ends and a ratio of length to radius of gyration of 161, tested by Mr. Christie, at Pencoyd, had no appreciable deflection until the load almost reached 10 300 lb. per sq. in., at which point it deflected 0.03 in. It failed at 12 113 lb. per sq. in. Euler's formula for this ratio of length to radius of gyration and for a modulus of elasticity of 27 000 000 gives 10 300 lb. per sq. in. (*Transactions*, Am. Soc. C. E., Vol. XIII (1884), p. 111.)

The Watertown Arsenal Report for 1910, p. 172, describes a steel column, 13 ft. 8¾ in. long, composed of one web 10 by ⅜ in., and four angles, 4 by 3 by ⅜ in., tested with flat ends, which had no deflection up to and including 32 000 lb. per sq. in. At 34 000 lb. per sq. in., the deflection was 0.17 in. and at 34 710 lb. per sq. in., it failed by deflection. The ratio of length to radius of gyration was 100. The yield point, as shown by tests of three similar columns with a ratio of 12, was 36 000 for this grade of steel.

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TABLE 2.—Yield Point in Tension and Compression, and Failure in Compression.

All bars from the same blow of Bessemer steel.

All specimens of each size from the same bar, and all of full size, "as from the rolls," except the tension tests of 3-in. flats.

All compression specimens tested with flat ends.

The specimens used for determining the yield points in compression were two diameters long.

All results are given in pounds per square inch.

Compiled from tests made by the late Charles A. Marshall, M. Am. Soc. C. E., and given in *Transactions,* Am. Soc. C. E., Vol. XVII (1887), p. 68, Tables 1 and 2.

Shape of bar and diameter, in inches.	Yield Point.				Failure Loads in Compression. Ratios of Length to Radius of Gyration are Given in Columns Headed $\frac{l}{\rho}$.							
	Tension.		Compression.		1 Test each.		1 Test each.		1 Test each.		1 Test each.	
	No. of tests averaged.	Load.	No. of tests averaged.	Load.	$\frac{l}{\rho}$	Load.	$\frac{l}{\rho}$	Load.	$\frac{l}{\rho}$	Load.	$\frac{l}{\rho}$	Load.
¾ Round.	4	46 090	1	47 300	50	44 980	..	.....	..	.....	..	.....
1 "	4	44 202	2	46 020	48	44 000	..	.....	..	.....	..	.....
1¼ "	3	40 747	2	43 460	48	40 850	..	.....	..	.....	..	.....
1½ "	3	40 275	2	41 290	48	41 880	..	.....	..	.....	..	.....
1¾ "	3	40 017	2	42 075	48	39 950	..	.....	..	.....	..	.....
2 "	3	38 207	2	38 830	48	40 850	..	.....	..	.....	..	.....
2¼ "	1	37 000	2	38 125	48	36 790	72	36 580	..	.....	..	.....
2½ "	1	36 100	2	36 840	48	35 650	70	34 450	..	.....	..	.....
¾ Square.	3	44 273	2	43 845	45	44 960	..	.....	..	.....	..	.....
$\frac{8}{10}$ "	2	47 815	2	49 055	..	.....	..	.....	..	.....	..	.....
1 "	3	43 560	1	44 025	42	43 080	..	.....	..	.....	..	.....
1¼ "	3	41 060	2	42 300	42	40 060	..	.....	..	.....	..	.....
1½ "	3	39 317	2	42 740	42	38 420	..	.....	..	.....	..	.....
1¾ "	3	38 193	2	40 630	42	39 450	..	.....	..	.....	..	.....
2 "	No test		1	39 870	42	39 750	..	.....	..	.....	..	.....
2¼ "	1	38 310	2	39 940	42	39 270	63	37 330	..	.....	..	.....
3×⅜ Flat.	3	47 363	..	.....	42	47 650	63	46 420	83	46 150	..	.....
3×½ "	3	44 417	..	.....	42	41 650	..	.....	83	43 490	104	42 170
3×¾ "	3	41 447	..	.....	42	43 580	..	.....	..	.....	102	40 700
3×1 "	3	39 397	..	.....	42	41 200	..	.....	83	36 920	..	.....
3×1¼ "	4	38 482	..	.....	39	38 350	63	35 430	..	.....	..	.....
3×1½ "	4	37 820	..	.....	42	36 920	..	.....	..	.....	..	.....
3×1¾ "	4	35 917	..	.....	42	35 760	..	.....	..	.....	..	.....
3×2 "	4	39 302	..	.....	42	37 670	..	.....	..	.....	..	.....
4×½ " *	2	53 800	..	.....	42	55 420	63	55 110	83	55 320	104	51 210
4×1 "	2	41 415	..	.....	42	42 630	..	.....	..	.....	104	39 270
4×1¼ "	1	36 680	..	.....	42	39 300	..	.....	83	38 200	..	.....
4×1½ "	1	37 580	..	.....	42	39 500	63	39 850	..	.....	104	39 100

* "Bar finished at very low heat."

Note that the failures by compression occurred at or below the yield point, that the yield point of each bar was about the same in tension and compression, and that it varied greatly in different sizes.

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TABLE 3.—Compression Tests, at Watertown Arsenal, of Rolled Steel H-Sections. Nominal size, 6 by 6 in., 23.8 lb. per ft., Tested in Horizontal Position, with Webs Vertical.

The "elastic limit" per square inch in tensile tests of four specimens from flange and two from web is given as follows: From flange, 28 500, 31 840, 29 500, 30 490; average, 30 107. From web, 32 590, 31 850; average, 32 245.

Compiled from Watertown Arsenal Reports for 1908 and 1909.

Ratio of length to least radius of gyration.	Tested with Flat Ends.			Tested with Pin Ends. Pins Vertical, in Plane of Web, and 3 In. in Diameter.		
	Yield point, in pounds per square inch.	Ultimate, in pounds per square inch.	Initial horizontal deflection.	Yield point, in pounds per square inch.	Ultimate, in pounds per square inch.	Initial horizontal deflection.
25	30 000	43 110	None given	31 000	45 000	None given.
25	30 000	40 710	" "	29 000	44 000	" "
25	30 000	44 730	" "	29 000	46 000	" "
50	No yield point before ultimate was reached.	28 980	" "	.....	28 850	0.08, 0.11 in.
50		28 620	" "	29 000	33 350	Kinky + 0.04 in.
50		29 000	" "		30 780	0.04, 0.03 in.
75		29 000	" "	No yield point before ultimate was reached.	28 000	0.07, 0.4 in.
75		29 000	0.04 in.		28 000	0.20, 0.12 in.
75		28 630	0.04 in.		28 650	0.07, 0.00 in.
100		28 000	wavy		24 000	0.23, 0.18 in.
100		26 000	0.00 in.		26 000	0.15 in.
100		28 000	0.00 in.		27 000	0.07, 0.10 in.
125		27 000	0.08 in.		22 540	0.10 in.
125		26 000	0.10 in.		23 925	0.20, 0.15 in.
125		25 000	0.17 in.		25 000	0.05, 0.06 in.
150		24 670	0.03 in.		20 360	0.15 in.
150		23 000	0.00 in.		17 130	0.16 in.
150		23 000	0.17 in.		10 100	0.42 in.

Note the great importance of the yield point in limiting ultimate capacity.

temporary safeguard against disaster under excessive loads, even though the columns would be irreparably injured. This justifies a somewhat higher working load for such columns than would otherwise be the case.

Short wrought-iron and steel columns of such solid or thick material, however, are not much used. Most short columns are made of such thin material, have their cross-sections so distended, or have some detail so weak, that if tested to destruction, failure would occur from local bending (or, as the author aptly terms it, secondary flexure), or from the giving way of some detail, under a load not much, if any, greater than the yield point.

The yield points of wrought iron and steel are, beyond question, the critical stresses for wrought-iron and steel columns, and should be

ubstituted for the crushing resistances given in Table 1 as "the trength to compression." Mr. Prichard.

In this connection, it is useful to know that the yield point of steel and probably of wrought iron) is practically the same, for metal of he same quality, in compression and tension. This is illustrated in 'ables 2 and 4 (after allowing for the fact that the yield points in ompression, in Table 2, were determined from very short specimens, hich tends, whether the stress be compression or tension, to raise some-hat the yield point), and many other cases could be cited if necessary.

'ABLE 4.—Comparative Tests in Tension and Compression of Specimens, 1¼ in. in Diameter, from Open-Hearth, Steel Bars of Ten Grades.

pecimens turned to 1.0092 in. in diameter. Compression specimens 12 in. long, with ratio of length to radius of gyration of 48.
Compiled from Watertown Arsenal Reports for 1886 and 1887.
All loads in pounds per square inch.

Mark.	Composition.			Tension.		Compression.	
	C., per cent.	Mn., per cent.	Si., per cent.	Ultimate.	Yield point.	Yield point.	Ultimate.
833	0.09	0.11	.....	52 475	30 000	30 500	32 125
123	0.20	0.45	.....	68 375	39 500	37 000	39 190
782	0.31	0.57	.....	80 600	46 500	44 500	45 500
795	0.37	0.70	.....	85 100	50 000	47 000	50 875
803	0.51	0.58	0.02	98 700	58 000	57 000	58 000
797	0.57	0.93	0.07	117 400	56 000	57 000	65 500
823	0.71	0.58	0.08	116 750	56 000	56 000	65 440
750	0.81	0.56	0.17	149 600	73 000	76 500	87 750
756	0.89	0.57	0.19	141 290	76 000	77 500	84 125
334	0.97	0.80	0.28	152 550	80 000	83 500	91 500

Note that the yield point in tension and compression is about the same for each grade, nd that failure of compression specimens of the five lowest grades occurred near the yield oint.

Digressing, somewhat, from the direct theme of this discussion: Some ngineers, in discussing the question of local slenderness or weakness, ave advocated the doctrine that columns should be designed so that ney cannot fail from local slenderness or weakness, but will, if tested, evelop the full strength of the column considered as a whole. This entirely a matter of economy and convenience, and of having ample iffness and sufficient strength to carry the load required with an mple margin of safety. If the column design is good in these respects, here is no reason whatever for changing to a less economical or con-enient form of cross-section. The cross-section usually has to be esigned with reference to connections, method of loading, and other ractical considerations, and, when thus designed, is likely to have ction and dimensions which, though amply strong and stiff, will not

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develop the strength of the column, as gauged by its length, radius of gyration, and cross-sectional area, unless sectional area not needed for any other purpose is added. The important consideration is that the strength of a column shall be gauged by its weakest feature.

Reverting to a direct discussion of the paper: The Rankine formula is not a satisfactory one for practical use. It cannot be made to agree well with the results of tests of wrought-iron and steel columns of moderate length, and, when the constants are adjusted to make the formula agree as well as may be with the results of tests of such columns, it gives results much too high for long columns. On the other hand, if, by suitable constants, the formula is made to give proper values for long columns, it gives results unnecessarily low for those of moderate length, and, with the same constants, may give results altogether too high for short columns, as is the case with the author's adaptation of the formula, as gauged by his own experiments. These facts are shown in Table 5, in which it is also shown that, when an allowance is made for slight unintentional eccentricity, to the extent recommended by A. Marston,* M. Am. Soc. C. E., the author's recent experiments confirm with remarkable closeness the theory of eccentrically loaded columns. The comparison is all the more striking and convincing from the fact that the experiments were made by an advocate, on theoretical grounds, of the Rankine formula, who is, presumably, free from any prejudice in favor of the theory which, with some variation in detail, is supported by the analyses of Messrs. Marston, Cain, Moncrieff, Johnson, Jonson, and others who have discussed the subject of columns before this Society.

Marston's equation for nominally centrally loaded columns, and the close approximations to Marston's equation which others have derived, are tedious to apply, but they can be closely approximated as to results by easily applied empirical formulas. The following empirical formulas agree well with the results of the author's experiments on solid mild-steel columns, $\frac{1}{2}$ in. in diameter, except in the case of very short ones, which failed at loads above the yield point. (These loads were, doubtless, much higher than those which the columns could permanently sustain without undue deformation).

For ratios of length, l, to radius of gyration, ρ, of 100 or less,

$$p = 58\,000 - 3\left(\frac{l}{\rho}\right)^2 \qquad (a)$$

For ratios of length to radius of gyration of 100 or more,

$$p = \frac{300\,000\,000}{\left(\frac{l}{\rho}\right)^2 + 7\,\frac{l}{\rho}} \qquad (b)$$

p being the load per square inch of cross-sectional area.

* *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), pp. 108-113.

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TABLE 5.—COMPARISON OF LOADS CAUSING FAILURE OF SOLID MILD-STEEL ROUND-ENDED COLUMNS, $\frac{1}{2}$ IN. IN DIAMETER, TESTED BY W. E. LILLY; AS SCALED FROM FIG. 4, WITH RESISTANCE INDICATED BY THEORY, RANKINE'S FORMULA, AND EMPIRICAL FORMULAS, *a* AND *b* (Equations *a* and *b*).
All results are given in pounds per square inch.

Ratio of length, l, to radius of gyration, ρ.	THEORY (MARSTON*). $p = \frac{58\,000}{1 + \frac{e\,y}{\rho^2} \sec. \left(\frac{1}{2}\frac{l}{\rho}\sqrt{\frac{p}{E}}\right)}$ y = distance from axis to extreme fiber. e = unintentional eccentricity. $e = 0.0$	$e = 0.00375$ in.	Results of W. E. Lilly's experiments; as scaled from Curve *A* of Fig. 4.	RANKINE'S FORMULA. As given by W. E. Lilly. $p = \frac{80\,000}{1 + \frac{\left(\frac{l}{\rho}\right)^2}{\frac{\pi^2 E}{80\,000}}}$	As arranged for moderate lengths. $p = \frac{62\,000}{1 + \frac{\left(\frac{l}{\rho}\right)^2}{10\,000}}$	Equation *a* $p = 58\,000 - 3\left(\frac{l}{\rho}\right)^2$ Equation *b* $p = \frac{300\,000\,000}{\left(\frac{l}{\rho}\right) + 7\,\frac{l}{\rho}}$
						a
20	58 000	54 400	61 500	72 200	59 600	56 800
40	58 000	53 200	53 500	55 800	53 400	53 200
45	58 000	52 600	52 000	51 700	51 600	51 900
60	58 000	49 400	49 500	40 500	45 600	47 200
70	58 000	45 300	46 000	34 400	41 600	43 300
80	46 200	39 300	41 500	29 300	37 800	40 800
90	36 500	33 000	34 500	25 300	34 300	33 700
100	29 600	27 600	28 000	21 600	31 000	28 000
						b
120	20 600	19 800	19 700	16 350	25 400	19 700
140	15 100	14 700	14 500	12 700	20 900	14 500
160	11 600	11 400	11 200	10 100	17 400	11 200
200	7 400	7 320	.....	6 770	12 400	7 200
300	3 290	3 270	.....	3 160	6 200	3 260
400	1 850	1 846	.....	1 810	3 650	1 840

*First published by A. Marston, M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), pp. 108-113, and since endorsed by many authorities. Marston used a value of 0.06 for $\frac{e\,y}{\rho^2}$ in comparing his formula with Tetmajer's tests of mild steel. This value applied to $\frac{1}{2}$-in. round bars gives $e = 0.00375$ in., as above.

Note that Rankine's formula cannot be made to agree with the "Experiments," while "Theory" agrees very closely when slight eccentricity is assumed.

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These equations are arranged for the sole purpose of formulating the author's experiments on solid mild-steel columns, ½ in. in diameter, and, as arranged, are presented without recommendation for any other purpose. When made entirely general for any grade of steel or wrought iron, they become

$$p = \text{yield point} - \text{constant} \left(\frac{l}{\rho}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots (c)$$

$$p = \frac{\pi^2 \times \text{modulus of elasticity}}{\left(\frac{l}{\rho}\right)^2 + x\left(\frac{l}{\rho}\right)} \quad \ldots\ldots\ldots\ldots\ldots (d)$$

The range of application of Equation *c*, and the value of its constant, have to be chosen to suit the series of columns for which the formulas are arranged. The value of x has then to be found by first determining the value of p for the length of column at which the application of Equation *c* ends and Equation *d* begins.

Equation *c* is a parabola; a form which the late J. B. Johnson, M. Am. Soc. C. E., advocated for short columns and those of moderate length. He, however, made his parabolas always tangent to Euler's curve, and advocated Euler's formula for long columns. On this basis, the constant for columns with pivoted ends is always

$$\frac{(\text{yield point})^2}{4\,\pi^2 \times \text{modulus of elasticity}}.$$

The writer prefers to increase the constant and modify Euler's formula for long columns, as per Equation *d*, in order to make a greater reduction for length. For bridges and buildings, long columns are usually tabooed, but for some kinds of construction they are not objectionable.

Professor Johnson compared his column formula with the very careful tests made by M. Considère. The tests with which the formulas were compared were of columns consisting of rectangular steel bars with pivoted ends, adjusted so that there was no appreciable eccentricity. The bars were of six degrees of hardness, and the results when plotted agreed with the corresponding six parabolic curves of the formula to an extent that was truly remarkable.

To show that the strength of a column, unless it is very short, is no function of the ultimate strength of the material, either in tension or compression, M. Considère cold-rolled the medium-hard steel, No. 5, which had a yield point of 47 000 lb. per sq. in., until it had elongated 10% of its original length. This raised its yield point to 71 000 lb. per sq. in., while its ultimate tensile strength was raised only from 83 000 to 88 500 lb. per sq. in. The columns of this steel, No. 6, with the yield point of 71 000 lb. and an ultimate of 88 500 lb. per sq. in., were stronger (in the maximum case more than 10% stronger) than the columns of steel, No. 8, with a yield point of 64 000 lb. and an ultimate strength of 98 000 lb. per sq. in.

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Professor Johnson also showed that his parabolic formula agrees well with other tests, notably Tetmajer's.* His (Johnson's) diagrams, comparing his parabolic formula with Tetmajer's tests, were reproduced by Mr. Marston,† with the locus of his (Marston's) formula for eccentrically-loaded columns added, in his discussion of Professor Cain's paper on the "Theory of the Ideal Column." They were again published, as given by Marston, in Professor Cain's discussion‡ of the paper on "Safe Stresses in Steel Columns," by J. R. Worcester, M. Am. Soc. C. E.

These diagrams are very interesting and instructive. They show a close resemblance between tests and theory when provision is made in theory for the inevitable initial deviation between the axis and the line of thrust; that a parabolic, empirical formula for short columns and those of moderate length is about as good and much simpler than the theoretical one; and that the averages of the tests agree well with the parabolic formula advocated by Professor Johnson; but they also show that a curve which gives more reduction for length accords better with the minimum results.

This was one of a number of facts which influenced the writer, in discussing Mr. Worcester's paper,§ to advocate a formula which gives a greater reduction. The formula, the reasons for advocating it, the method of using it in cases of intentional eccentricity, the additional limitations suggested for short columns, and other practical and theoretical considerations were outlined in this discussion and need not be repeated here, but it is best to add a caution with regard to the yield point.

The yield point varies greatly, even in steel from the same melt, as shown in Table 3, and will cause great differences in the strength of columns of ordinary lengths. This is especially important in dealing with columns made of thick plates and large, thick angles, which are likely to have much lower yield points than thinner and smaller sizes of nominally the same grade of steel. It will not do, in such cases, to pin one's faith unreservedly to any formula, no matter how well it is supported by tests of solid bars, ½ in. in diameter, or other small shapes.

Such tests, however, are very useful in determining experimentally the laws of columns by systematically varying one condition while others remain constant. The author's tests are to be commended, in this regard. To plat columns differing greatly in yield point, eccentricity, end restraint, and local stiffness on the same diagram in a kind of hodge-podge, only leads to confusion and tends to discredit the

* The information regarding Professor Johnson's parabolic formula and M. Considère's tests is taken from Professor Johnson's "Materials of Construction," pp. 361-363, and "Modern Framed Structures," pp. 148-152.

† *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), pp. 108-113.

‡ *Transactions*, Am. Soc. C. E., Vol. LXI (1908), pp. 204-205.

§ *Transactions*, Am. Soc. C. E., Vol. LXI (1908), pp. 165-178.

Mr. Prichard. valuable information which has been acquired by analytical and experimental investigation. The value of rational investigation, in which sound theory explains and supplements experiments and experiments put to the test and verify sound theory, should receive general recognition.

An important step in this direction will have been taken when it becomes generally recognized that Rankine's formula is based on a blunder caused by a plausible but fallacious assumption of analogy in deflection of beams and columns, that it has only the semblance of a "theoretic basis," and that it is not even a good empirical formula for wrought iron and steel.

Mr. Branne. J. S. BRANNE, M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest, and has been especially attracted by the attempt to find the size of lace-bars to connect the component parts of the columns, in order that said parts may act truly as one piece in the distribution of loading, which produces, generally, both direct compressive stresses and bending stresses, as it may be; and that even wide columns, meaning those of a low ratio of $\frac{l}{r}$ have some side deflection due to compressive loading, speaking now of such deflection in an "ideal column."

That there is a natural irregularity and deviation from a straight line, due to lack of uniformity in the component parts, is well known to all engineers, this very thing making the theoretical determination of strength so hard to reconcile with the data as actually revealed by tests.

It seems to the writer that there are two kinds of compression members: the wide ones and the slender ones; the first failing by flow of metal, directly compressed, the second by flow and flexure stresses, which latter are induced by the sidewise deflection of the compression member. By failing is here meant "a perceptible set, showing unfitness for further loading, not actual destruction."

Experiments have shown that wide columns, when properly fabricated, fail when the nominal unit stress approaches or ranges around the elastic limit. The experiments made by Mr. James E. Howard* show test results on wide columns. Just where to draw the line between "wide" and "slender" columns is not so easy. The Progress Report of the Special Committee on Steel Columns and Struts† gives a series of diagrams of tests, which are condensed into one diagram, Fig. 23, showing the results of many tests. The curves are quite flat up to $\frac{l}{r} = 70$, after which they drop more sharply.

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* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 429.

† *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 401.

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The cross-sections of the columns in Mr. Howard's tests were about 90 sq. in., and the ratio, $\frac{l}{r}$, was 26 for Column No. 1, and 47 for Columns Nos. 2 to 5. Columns Nos. 1 and 2 did not fail; but Nos. 3, 4, and 5 did fail, Nos. 3 and 4 in the pin-plates, and No. 5 in the body of the column. The manner of failure in No. 5 was that the webs buckled, resulting in a "sharp bend, or set"; Mr. Howard's Fig. 1, Plate XLII, shows a photograph of Column No. 5 after failure.

In none of these columns was the failure induced to any perceptible degree by flexure stresses, as such combined action of direct compression and bending would have caused failure somewhere below the elastic limit.

It must be remembered here that the test pieces of the component parts showed quite a variation in tensile strength, which makes it impossible to refer to a certain magnitude as the elastic limit of the column, as a whole. Columns Nos. 3 and 4 failed at points where the punching of many holes (with much subsequent riveting) induced numerous and complex local stresses; No. 5 failed by the buckling of the web at a point 12 ft. 8 in. from the end, probably due to local weakness, for the test load gave a nominal average unit stress of 30 490 lb. per sq. in. In these columns the lace-bars could not have had much to do before the elastic limit of the column was reached, when locally weak spots put them into activity. It does not seem possible to calculate the lace-bars in wide columns which fail at unit stresses approaching or ranging around the elastic limit. If lacing is used in such columns, it should be quite heavy, in order to take care of local weaknesses; or, still better, lacing should be avoided, and one should use cover-plates or continuous diaphragms, forming part of the section of the column, so as not to waste material.

If heavy lacing is used, the tie-plates at the ends of the columns should be long and thin, rather than short and thick, to steady up the member as much as possible; and where the component parts are fairly wide, say 15 in. or more, diaphragms should be used at the ends, in addition to the plates, thus helping to equalize the stresses carried into the compression member. Similar precautions should be taken at intermediate points where loads are applied.

As to columns of the second kind, namely, the slender ones, all tests show that failure occurs much below the elastic limit, clearly indicating the presence of bending stresses in addition to the direct compressive stress. The bending will induce shear in the columns, producing compressive or tensile stresses in the lace-bars.

If the deflection be assumed to result from a uniform transverse loading, which assumption is not quite correct, it will be evident that the stresses in the lace-bars are greatest at the ends of a free-end column. If the column is not free to move at the ends, but stands

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between this type and the one with restrained ends, the shear is also greatest at the ends, necessarily, but it is evident that greater care should be taken in this latter type to tie the component parts together more securely at the ends than for free-end columns. If comparison be made, then, with uniformly-loaded beams, it will be noticed that in one with free ends the longitudinal stress caused by the maximum center bending moment, $+\frac{wl}{8}$, has to be transmitted from the compression side to the tension side in a length, $\frac{l}{2}$; in the one with restrained ends, the same amount has to be taken up (from $+\frac{wl}{24}$ at the center to $-\frac{wl}{12}$ at the ends), but this presupposes that the column is held rigidly at the ends, a bending moment, $-\frac{wl}{12}$, is already supposed to exist, and this should be taken care of by extra long tie-plates or diaphragms between the channels.

To proportion the lace-bars under the assumption of uniform transverse loading, a working formula has to be used, for example, American Bridge Company, medium steel, $17\,000 \div \frac{1}{11\,000}\left(\frac{l}{r}\right)^2$, or American Railway Maintenance of Way Association, $16\,000 = 70\,\frac{l}{r}$, and the reduction of fiber stress is assumed as the unit extreme fiber stress due to bending, whence the uniform load can be found.

On this basis of approximating the real conditions, which certainly are unknown, it will be found:

(1) When the column is slender, yet the section not great, the average specification covers the size of lattice-bars very well.

(2) When the column section becomes quite heavy, and, at the same time, due to space requirement of some kind, the component parts, generally channel-shaped, are placed closely together, making failure evident in the lace-bars, it will be found that the generally used lace-bars, as given in the specifications, are not strong enough, but must be increased, finally resulting, economically, in doing away with lacing and substituting cover-plates, or a continuous diaphragm, as noted before.

Finally, as regards the wide column, while the lace-bars can have nothing to do from a theoretical sidewise deflection, at the same time, one component part may be stressed more than the other, due to a

faulty foundation or a small error in fabrication, causing an uneven unit stress, which produces stresses in the lacing. As long as the component parts are small, such unevenness can be cared for by the lacing taking up the longitudinal shear; but, when the component parts are very heavy, the lacing cannot make them act as one, for, being proportionally so much lighter than the component parts, they become overstrained and yield. Cover-plates or continuous diaphragms cannot, of course, overcome the faults of foundation or errors in fabrication, but have the strength to transmit unevenness of stress without becoming overstrained, thus making the column act as one homogeneous piece, reducing the unevenness of stress in the component parts. Mr. Branne.

The tests conducted by Arthur N. Talbot, M. Am. Soc. C. E., at the University of Illinois Engineering Experiment Station,* showed an irregular action of the lace-bars, some of the bars showing more stress near the center than at the ends. Further data bearing on these tests may be found in the many interesting notes in the same bulletin.

W. E. Lilly, Esq. (by letter).—Mr. Godfrey's discussion contains a great deal of controversial matter regarding the strength of columns, and the writer will endeavor to reply to some of the points that have been raised. It is to be regretted, however, that Mr. Godfrey was not more definite in stating his objections. Vague statements, unless based on reasoning of some kind or experimental evidence, can hardly be considered as discussion on the subject. Mr. Godfrey states that he has repeatedly assailed both the Rankine-Gordon and Euler formulas for columns, and has observed and demonstrated, in his practice and reading, that confidence in these formulas has resulted in failure. The writer totally differs with him, after an examination of the majority of the tests made on columns. From the results of his own experiments, he has come to the conclusion that the Rankine-Gordon formula, when secondary flexure is allowed for, may be relied on for the design of columns; and, as he has shown in previous papers, it gives results which agree more closely with the experimental tests than can be obtained by the use of any other formula; further, the formula is logically derived, a merit not possessed by any other column formula. He has fully discussed the derivation of both the Rankine-Gordon and Euler formulas in the papers referred to in the Appendix. Mr. Lilly.

Mr. Godfrey's examples of the uses of Rankine-Gordon's and Euler's formulas are unfortunate, and show how dangerous it is to use the constant in the Rankine-Gordon formula without knowing how it has been derived. Euler's formula always gives a greater value than that obtained from the Rankine-Gordon formula under similar conditions of column loading. The writer would refer Mr. Godfrey

* *Bulletin No. 44*, June 6th, 1910.

Mr. Lilly. to the constants given in the paper and to the manner of their derivation.

Mr. Godfrey states that the Rankine-Gordon formula fails to meet the needs of the practical design of columns, and that the straight-line formula is far superior. It gives results closer to those obtained from experiments, and there are several reasons why it should. In the paper the writer has referred to the merits of the Rankine-Gordon formula as a practical one for the design of columns, and does not propose to discuss it further. He has also referred to the straight-line formula. His own experiments show that it does not give results which agree closely with the experiments, and there are no reasons, based either on theory or experimental evidence, why it should.

Mr. Godfrey mentions two instances of columns, in which the component parts were joined by batten-plates and of the disastrous results which followed the use of these columns. The writer agrees with Mr. Godfrey that lattice bracing should be used, as columns of this kind cannot transmit the shear stresses set up by the bending of the columns.

Mr. Eckersley's remarks are generally in accordance with the writer's views; the mathematical treatment of the column problem, however, in the present state of knowledge, appears to be so full of difficulties as to be almost incapable of rigorous solution, notwithstanding the efforts made in this direction by various investigators. It is for this reason that so many semi-empirical formulas have been put forward at different times, and of these, as stated in the paper, the writer considers the Rankine-Gordon formula, together with his own modification of it, to be the best.

He thinks that any attempt to solve the column problem should be approached somewhat as follows: Let a solid column, as shown by Fig. 1, be given, in which the load is applied at a given eccentricity, e. When loading is gradually applied to the column, it commences to bend or deflect, and the magnitude of the deflection depends on the elasticity of the material, other quantities being assumed as constant. As the loading is increased, the deflection increases, and when it reaches a limiting value, depending on the strength of the material, failure of the column takes place. The problem thus considered shows that, among other quantities, three are involved, namely, the coefficient of elasticity, E, the strength of the material, f, and the deflection, d, when failure of the column takes place.

It follows that any column formula must of necessity, among other quantities, include these three, if it is to be generally applicable. The mathematical treatment of the problem leads to difficulties in determining the deflection, d, and as far as the writer is aware, it can only be approximated to. It remains for future investigators to determine the rigorous form of the function governing the deflection.

The formation of a satisfactory column formula will then be an easy matter. Mr. Lilly.

Mr. Prichard's remarks require a more detailed reply, and the writer will endeavor to discuss the more salient points raised. Mr. Prichard states that he "does not share the author's opinions as to the theoretical and practical excellence of the Rankine formula, and he does not approve of the use of such a high unit stress for the 'strength to compression'"; and, further, after referring to Rankine, he states:

"Subsequently, he [Rankine] makes the mistake of applying one of these equations to the deflection of columns, entirely overlooking or neglecting the fact that, in all cases of ordinary columns, part, at least, of the stress in the extreme fiber is due to direct compression. This mistake leads to an erroneous formula for columns, which, on the strength of Rankine's endorsement, has deceived and has continued to deceive engineers, and interfere with a correct understanding of the subject, for more than fifty years.

"The error is reflected in the author's Equation 6 for deflection of columns, which he makes a factor of the total stress intensity in the extreme fiber, f, instead of a factor of $f - p$, as it should be (p being the load per unit area). The author prefaces this substitution with an unwarranted assumption, which, as he states, 'errs', and gives what he terms 'the first approximation'."

With regard to the foregoing remarks, either Mr. Prichard is in error, or he has not studied the underlying assumptions made in the derivation of the Rankine-Gordon formula; for the real point of the formula is that it does allow for the stress in the extreme fiber due to direct compression. Mr. Prichard is also in error in stating that $f - p$ should be used for p in the derivation of the formula, and the writer would refer him to the articles in *Engineering,* quoted in the appendix to the paper, in which he will find the Rankine-Gordon and Euler's formulas fully discussed. There is no doubt that, after perusing the same, he will qualify his opinion regarding the substitution referred to being an unwarranted one. Mr. Prichard then quotes from Rankine the theorem underlying the Eulerian analysis of a spring or slender column, and states:

"In view of the prevalent uncertainty as to the theory of perfectly elastic, initially straight, centrally loaded columns, the writer suggests that the author state whether he accepts this theorem of Rankine's; and if he does not accept it, that he indicate wherein he considers Rankine's 'proof' to be in error."

In the articles in *Engineering* previously referred to, the writer has already stated his objections to the Eulerian analysis, and pointed out wherein it errs; the assumption of I, the moment of inertia, remaining constant when applied to columns is false; also the value of E does not remain constant outside the range of the elastic limits. An article

Mr. Lilly. in *Engineering** on the strength of columns will give Mr. Prichard further information on this point.

Mr. Prichard does not agree with the writer's high unit stresses adopted for the strength to compression, and states:

"The yield points of wrought iron and steel are, beyond question, the critical stresses for wrought-iron and steel columns, and should be substituted for the crushing resistances given in Table 1 as 'the strength to compression'."

The writer does not agree with this statement; from his own experiments he is of the opinion that the yield point can only be determined with any degree of accuracy on annealed bars. On bars which had been subjected to cold rolling or to permanent strain he doubts whether any real yield point could be said to exist. He thinks that too much importance has been attached to this quantity as a standard measurement. Tables 2 and 3, given by Mr. Prichard, show a variation of the values from 29 000 to 49 000 lb. per sq. in., and, in view of such wide variations, the writer prefers to retain the values for the strength to compression given in the paper. He would refer Mr. Prichard to a paper on the elastic limit and strength of materials,† in which the above points are more fully discussed.

Mr. Prichard refers to the formula for columns given by Marston, and states, in Table 5, that this formula was first given by Marston in the *Transactions*, Am. Soc. C. E., Vol. XXXIX, and since endorsed by many authorities. Mr. Prichard is in error; the formula was given, as long ago as 1878, by Smith,‡ and modifications of it have been given by Perry and Neville. Mr. Prichard puts forward this formula as a rigorously theoretical one; this, however, is not the case, as the writer showed in the articles in *Engineering* already referred to. Considered from the point of view of a practical column formula, it is open to many objections. The values given in Table 5 show this. In the particular case when the eccentricity, $e = 0$, the loads on the column, for values of $\frac{l}{\rho}$ between 0 and 70, are all equal, and, for values greater than 70, the loads follow the Eulerian curve; this is equivalent to assuming that, for values of $\frac{l}{\rho}$ between 0 and 70, the column has no deflection when failure takes place, and then for values of $\frac{l}{\rho}$ greater than 70 for the deflection to vary approximately as the square of the length of the column. This is contrary to experience, as shown by tests on columns, and is evidently incorrect. To assume some value

* August 23d, 1912.

† Read before the Institution of Civil Engineers of Ireland, December 6th, 1911.

‡ *Proceedings*, Edinburgh and Leith Engineering Society.

for the yield point of the material from experiment, and then to assert that no column within the range of $\frac{l}{\rho}$ between 0 and 70, when loaded with the load per unit of area, will bend, is inconceivable; and yet this is the standpoint from which Mr. Prichard bases his faith in the Smith-Marston formula. Experiments show that columns always bend before failure takes place, and the writer is of the opinion that some function of the length must be assumed for the deflection in the derivation of any useful column formula. It is in this respect that the Eulerian analysis has had such a retarding influence on investigations on columns, in the enunciation of a critical load as obtained from Euler's formula, together with a zero deflection of the column, in contradistinction to the idea that the essential point to be considered is the deflection of the column when failure takes place. The Smith-Marston formula has never found favor with engineers, owing to its being troublesome to use in practice; if it were theoretically correct, this would be no argument against it, but, as it is not, there is nothing to be gained in using it. In conclusion, Mr. Prichard states: Mr. Lilly.

"An important step in this direction will have been taken when it becomes generally recognized that Rankine's formula is based on a blunder caused by a plausible but fallacious assumption of analogy in deflection of beams and columns, that it has only the semblance of a 'theoretic basis,' and that it is not even a good empirical formula for wrought iron and steel."

The statement of the derivation of the Rankine-Gordon formula is incorrect; if anything, it applies more nearly to the formula advocated by Mr. Prichard, as the writer has shown in the articles already referred to.

Mr. Branne's remarks are of interest to the writer. As far as he is aware, the Rankine-Gordon formula is the only one which permits the determination of the sizes of the bracing in a logical way, and the examples given in the paper show that the results obtained by its use are closely correct. From a comparison of the tests carried out on columns, it is difficult to generalize, owing principally to the limited range of the length of the columns tested, and it is to be hoped that, in the future, in arranging for further experiments, this point will be borne in mind.

In conclusion, the writer thanks those gentlemen who have discussed the paper. Some difference of opinion was to be expected, the subject being one of much complexity. If the paper leads to a co-ordination of the results of the tests on columns, the writer will feel that he has been amply repaid for the labor of preparing it.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1249

THE FLOOD OF MARCH 22D, 1912, AT PITTSBURGH, PA.*

By Kenneth C. Grant, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Le Conte, William R. Copeland, Jean de Pulligny, J. Waldo Smith, Morris Knowles, and Kenneth C. Grant.

On March 22d, 1912, the Allegheny and Monongahela Rivers, which join at Pittsburgh to form the Ohio, reached a gauge height of 28.1 ft., the highest stage since February 16th, 1908, when there was a 30.7-ft. flood.† Except for a second flood, on March 20th, 1908, which reached a stage of 27.3 ft., no other flood of importance had occurred at Pittsburgh for four years, although the danger line, 22 ft., had been reached several times, the highest stage occurring on January 31st, 1911, when the rivers rose to a gauge height of 25.2 ft. The record stage of 35.5 ft. occurred on March 15th, 1907, when about $6 000 000 of flood damage resulted.

In view of the exhaustive studies looking toward methods of flood relief that have been made by the Flood Commission of Pittsburgh, which has recently published its report, it is thought that a brief description of the recent flood will prove of interest. To this description has been added a graphical study of the peak reduction that could have been obtained if the system of storage reservoirs proposed by the Flood Commission had been in operation.

* Presented at the meeting of November 6th, 1912.

† The portion of this paper describing the flood was in large part written directly after its occurrence, and published in *Engineering News* of April 4th, 1912. Since this first writing, additional data have become available, and the description has been modified and enlarged accordingly.

At 8 A. M., on March 21st, 1912, the gauge at Pittsburgh registered 17.9 ft. For 5 days prior to this date the rivers had been flowing fairly full, the gauge readings at Pittsburgh varying between 13 and 18 ft. The weather had been mild, and the high temperatures that prevailed for part of the time had gradually brought the ice out of the tributaries and head-waters of the Allegheny. The ice on the Monongahela and its tributaries had gone out some time before. Had cold weather continued until the 20th, when the flood rainfall began, the ice on the Allegheny would have been held back until then, the flood would have been considerably higher, and might have been dangerously increased by ice gorges. The ice moved out of the Upper Allegheny and its tributaries on the 17th. On the Clarion River a heavy gorge formed below Clarion, raising the stage at that place to 13.0 ft. on the 16th, which rapidly reduced to 4.6 ft. on the 17th, when the ice moved out early in the morning. Ice was running on the Allegheny at Parker on the 16th and 17th, and began passing Freeport on the morning of the 17th.

During the 21st the rivers at Pittsburgh rose rapidly and crossed the danger line, or the 22-ft. stage, about 2 P. M., rising steadily until 5 A. M. on the 22d, when the maximum of 28.1 ft. was reached. The crest remained at this level for 2 hours and then dropped at about the same rate as it had risen, falling below the danger line at about 2 A. M. on the 23d, and registering 19.6 ft. at 8 A. M. The rise was probably checked to some extent by a sudden drop in temperature on the 21st. The water was above the danger line for about 36 hours, during which time the total discharge above the 22-ft. stage was about 6 221 000 000 cu. ft. The discharge at the crest of 28.1 ft. was about 300 000 sec-ft.

All the rain causing this flood fell practically in the 12 to 14 hours preceding 8 A. M on the 21st, although in some sections it continued for a few hours after this time. After the middle of March, with the exception of an inch or two in the extreme northern part of the Allegheny Basin, there was no snow on the ground to contribute to the flood run-off.

The precipitation was lightest over the Northern Allegheny Basin, where it was mostly in the form of snow. The Upper Allegheny, therefore, contributed little to the rise, especially as practically all the 6 in. of snow which fell was still on the ground on March 23d. French Creek was at a fairly high stage, being above 10 ft. throughout the

20th and 21st. The Clarion, however, rose to only 6.6 ft., 9.4 ft. below record stage, and Red Bank, Mahoning, and Crooked Creeks were not large contributors, so that the Allegheny at Kittanning rose to only 17.2 ft., 12 ft. below record stage, at 4 P. M. on the 21st. At Freeport, the river registered 23.2 ft. at 6 P. M. on the 21st, when the water was rising at the rate of 0.1 ft. per hour. The main contributor from the Allegheny Basin was the Kiskiminetas River, which reached a stage of 28.0 ft. at Avonmore at 8 P. M. on the 21st, 9 hours before the crest was reached at Pittsburgh.

On the Monongahela Basin, the Cheat, Tygart Valley, and West Fork Rivers were not important factors in the rise. The Youghiogheny, however, was an important contributor, and, with the Kiskiminetas, was the main cause of the high stage attained at Pittsburgh. It reached a stage of 22.7 ft. at West Newton at 8 P. M. on the 21st. Except during the flood of March, 1907, when the maximum at West Newton was 28.2 ft., this river had never been higher than 22.0 ft. at this station. The time necessary for floods of these heights to move from West Newton and from Avonmore to Pittsburgh is from 9 to 12 hours, and both the Youghiogheny and Kiskiminetas, therefore, must have delivered their greatest discharge at Pittsburgh at about flood-peak time, between 5 and 7 A. M. on the 22d.

The high stages in the Kiskiminetas and Youghiogheny Rivers were to be expected, as the rainfall was heaviest over their basins. The maximum rainfall recorded during this storm was at Lycippus, Pa., on the Kiskiminetas Basin, where 3.05 in. fell in the 12 to 14 hours following the evening of March 20th. The next greatest rainfall was at Johnstown, Pa., on the same basin, where 2.91 in. fell during the same period. The region of greatest rainfall centered around these stations. Toward the north the recorded rainfall decreases rapidly in amount, and it is evident that, with the exception of the Kiskiminetas Basin, little of the Allegheny Basin received more than 1 in. of rainfall, while a considerable portion received only ½ in., or less. Toward the south the amount of rainfall recorded decreases more gradually to about 1 in. near Grafton and Parsons, W. Va., and farther south, near the head-waters of the West Fork, to less than ½ in. A considerable portion of the Monongahela Basin, therefore, mostly included within the Youghiogheny water-shed, received 2 in. or more of rainfall, while the larger part received more than 1 in.

The rainfall causing this flood is given in Table 1, and the map of the basins, Fig. 1, shows its distribution. Table 2 gives the daily gauge heights at stations on the main rivers and their tributaries before and after the time of the Pittsburgh peak, including time and height of maximum stage in so far as records exist.

TABLE 1.—RAINFALL CAUSING FLOOD OF MARCH 22D, 1912, AT PITTSBURGH.*

ALLEGHENY BASIN.		MONONGAHELA BASIN.		OHIO BASIN.‡	
Station.	Depth, in inches.	Station.	Depth, in inches.	Ohio Basin.	Depth, in inches.
Allegany, N. Y.	0.65	Buckhannon, W. Va.	0.30	Beaver Dam, Pa.	0.98
Angelica, N. Y.	0.92	California, Pa.	1.65	Claysville, Pa.	0.68
Baldwin, Pa.	0.64	Central Station, W. Va.	0.70	Davis Island Dam, Pa.	0.93
Bolivar, N. Y.	1.00	Confluence, Pa.	2.37	Greenville, Pa.	0.98
Brookville, Pa.	0.35	Creston, W. Va.	0.43	Grove City, Pa.	0.93
Clarion, Pa.	0.12	Elkins, W. Va.	0.68	Sharon, Pa.	0.82
Derry, Pa.	1.99	Fairmont, W. Va.	1.59	Ellwood City, Pa.	0.91
Franklin, Pa.	1.07	†Glenville, W. Va.	0.45	Beaver Falls, Pa.	0.96
Freeport, Pa.	1.61	Grafton, W. Va.	0.80	Coraopolis, Pa.	0.86
Haskinville, N. Y.	0.50	Greensboro, Pa.	1.54		
Herr Island Dam, Pa.	1.15	Greensburg, Pa.	2.53		
Hunt, N. Y.	0.40	Irwin, Pa.	1.64		
Indiana, Pa.	0.20	Lock No. 4, Pa.	1.19		
Johnstown, Pa.	2.91	Lost Creek, W. Va.	0.73		
Lycippus, Pa.	3.05	Mannington, W. Va.	1.50		
Olean, N. Y.	1.00	Morgantown, W. Va.	2.00		
Otto, N. Y.	0.40	Parsons, W. Va.	1.00		
Parker, Pa.	0.79	Philippi, W. Va.	0.71		
Pittsburgh, Pa.	0.58	Pickens, W. Va.	0.69		
Saegerstown, Pa.	0.85	Rowlesburg, W. Va.	1.56		
Saltsburg, Pa.	1.63	*Smithfield.	1.47		
Springdale	1.32	Somerset, Pa.	1.55		
*Volusia, N. Y.	0.50	Uniontown, Pa	1.65		
Warren, Pa.	0.66	West Newton, Pa.	2.19		
Westfield, N. Y.	0.39				

*This rain fell in the 12 to 14 hours preceding 8 A. M. on March 21st; in a few sections it continued for a few hours after this time.

†Just outside of basin.

‡Records used in locating isohyetal lines on Fig. 1.

Damage by Flood.—The damage caused by this flood in Pittsburgh and in other communities bordering the rivers above and below that city was considerable. A careful census of flood damage, made by the Pittsburgh Flood Commission after the 27.3-ft. flood of March 20th, 1908, showed a direct loss at Pittsburgh alone, for a flood about 1 ft. lower than the recent stage, of more than $400 000. It would seem reasonable, therefore, to place the direct loss in the recent flood at about $500 000, although, when complete information is at hand, this figure may be modified, especially as certain streets in the flooded district were raised during 1911. About 250 acres of the low-lying

Stream.	Station.	March.										Highest recorded stage.	
		15th	16th	17th	18th	19th	20th	21st	22d	23d	24th	25th	Stage.	Date.
Allegheny River	Red House	5.2	7.8	7.0	7.8	8.4	*a* 9.4	8.7	7.3	....	....	....	13.6	Mar. 2, 1910
Allegheny River	Warren	*F* 1.8	*F* 3.0	5.5	7.0	7.7	8.9	9.7	7.7	6.0	....	....	17.4	Mar. 17, 1865
Tionesta Creek	Nebraska	3.7	5.5	3.4	4.4	4.5	4.7	4.0	3.6	....	....	....	....	.....
Oil Creek	Rouseville	2.3	3.3	3.6	4.0	3.8	4.5	3.6	....	....	....	....	....	.....
No. Br. French Creek	Kimmeytown	3.7	5.5	6.6	8.9	6.7	8.5	6.0	4.3	3.5	....	....	13.9	.....
Cussewego Creek	near Meadville	5.0	....	9.7	10.5	10.4	10.3	9.9	9.4	8.5	....	....	....	.....
Sugar Creek	Wyattville	4.0	3.4	3.7	3.9	3.8	3.6	3.7	3.7	3.6	....	....	....	.....
French Creek	Carlton	5.5	8.2	6.9	7.8	9.2	10.5	*b* 10.6	....	....	....	....	16.5	1888
Allegheny River	Franklin	*F* 2.3	*F* 4.7	5.9	9.1	9.8	11.6	11.6	9.9	7.9	....	....	....	Mar. 17, 1865
Clarion River	Clarion	*F* 2.7	*c* 13.0	4.6	6.0	4.3	6.0	6.6	5.6	5.0	....	....	16.0	Mar. 20, 1905
Allegheny River	Parker	*F* 2.7	*F* 5.5	7.6	10.4	10.7	11.9	12.7	11.4	8.4	....	....	28.0	Mar. 1865
Allegheny River	Kittanning	....	9.5	16.4	13.7	14.1	15.5	*d* 17.2	14.9	....	....	....	29.3	Mar. 1865
Red Bank Creek	St. Charles	7.2	6.2	5.9	5.5	5.7	5.8	6.2	5.7	....	....	....	....	.....
Mahoning Creek	Furnace Bridge	7.5	6.6	5.5	5.5	5.2	5.6	*e* 6.4	7.2	....	....	....	....	.....
Crooked Creek	Hileman's Farm	5.6	3.2	3.8	3.3	2.8	2.4	*f* 6.5	5.3	3.6	....	....	....	.....
Black Lick Creek	Black Lick	7.2	6.9	5.4	5.5	5.4	5.6	*g* 10.1	8.0	....	....	....	13.2	Mar. 14, 1907
Conemaugh River	Johnstown	6.7	7.0	4.5	5.9	5.9	7.3	*h* 11.8	10.0	6.3	....	....	21.0	May 31, 1889
Kiskiminetas River	Saltsburg	3.1	8.5	4.5	4.3	4.3	4.3	*i* 8.5	12.0	5.8	....	....	22.1	1859
Kiskiminetas River	Avonmore	11.7	15.6	10.8	10.6	10.4	11.7	*j* 22.8	22.0	....	....	....	34.9	1859
Allegheny River	Freeport	8.0	14.0	14.4	12.3	14.7	15.5	*k* 19.3	22.2	15.5	....	....	32.7	Feb. 18, 1891
West Fork River	Enterprise	11.8	12.2	9.0	6.3	4.0	4.1	8.9	7.6	....	....	....	33.0	July 1888
Tygart Valley River	Belington	6.3	12.2	8.4	6.0	5.1	5.4	6.7	7.9	....	....	....	19.6	July 1888
Tygart Valley River	Fetterman	12.0	14.8	11.5	8.1	6.7	8.8	10.3	11.2	9.2	10.3	10.8	29.0	July 1888
Cheat River	Rowlesburg	5.6	9.0	5.6	4.0	3.7	4.4	*l* 6.0	6.7	5.2	....	....	22.0	July 10, 1888
Cheat River	Ices Ferry	7.8	9.3	6.3	5.4	5.0	6.2	*m* 9.5	8.3	6.3	....	....	....	July 10, 1888
Monongahela River	Fairmont	19.2	23.6	22.0	17.6	16.6	16.6	*n* 19.3	20.6	19.0	....	....	37.0	July 10, 1888
Monongahela River	Greensboro	12.8	20.9	15.3	11.8	10.3	10.5	*o* 16.3	18.3	13.9	....	....	39.0	July 10, 1888
Monongahela River	Lock No. 4	17.5	25.7	23.2	17.0	13.3	12.7	*p* 18.3	27.5	20.5	....	....	42.0	July 11, 1888
Youghiogheny River	above Confluence	10.9	9.6	6.1	5.5	5.3	7.2	15.3	10.2	6.5	....	....	16.8	Mar. 14, 1907
Youghiogheny River	below Confluence	4.4	8.3	5.0	4.4	4.3	5.4	9.8	10.3	6.0	....	....	18.6	Mar. 14, 1907
Casselman River	Confluence	10.5	8.8	5.1	4.9	4.7	7.0	17.0	9.6	5.6	....	....	18.1	Mar. 14, 1907
Laurel Hill Creek	Confluence	8.0	7.0	4.1	4.0	3.8	5.2	14.4	7.2	4.1	....	....	17.0	Mar. 14, 1907
Youghiogheny River	Connellsville	8.2	10.9	7.0	6.6	6.1	7.6	*q* 14.7	12.3	....	....	....	....	Mar. 14, 1907
Youghiogheny River	West Newton	4.9	14.1	8.3	6.0	5.3	5.8	*r* 11.8	19.8	9.9	....	....	28.2	Mar. 14, 1907
Ohio River	Pittsburgh	9.6	18.3	17.9	13.1	13.4	13.3	17.9	*s* 28.1	19.6	....	....	35.5	Mar. 15, 1907
Ohio River	Wheeling	12.8	19.7	27.0	25.5	21.4	21.0	21.9	*t* 33.3	38.4	31.4	25.5	53.1	Feb. 7, 1884
Ohio River	Parkersburg	15.7	23.4	26.2	28.0	26.0	23.6	24.0	29.0	34.8	*u* 36.5	35.8	53.9	Feb. 9, 1884
Ohio River	Cincinnati	37.2	42.6	44.0	45.4	46.6	47.0	46.5	47.0	45.2	45.8	*v* 50.2	71.1	Feb. 14, 1884
Ohio River	Cairo	34.4	34.9	....	38.2	40.4	42.4	44.1	45.4	46.4	....	48.5	*w* 54.0	Apr. 6, 1912

F. Frozen. *a*. Max. at noon, 9.65. *b*. Max. at noon, 10.7. *c*. Ice gorge; broke at 5 A. M. on 17th. *d*. Max. at 4 P. M., 17.2. *e*. Max. at 5 P. M., 8.4. *f*. Max. at 5 P. M., 9.4. *g*. Max. at 1 P. M., 11.1. *h*. Max. at noon, 13.6. *i*. 14.0 at 2 P. M.; still rising. *j*. Max. at 8 P. M., 28.0. *k*. 23.2 at 6 P. M.; rising 0.1 ft. per hour. *l*. Max at 2 P. M., 7.0. *m*. Max. at 5 P. M., 9.8. *n*. Max. at 6 P. M., 21.4. *o*. Max at 4 P. M., 20.0. *p*. 26.2 at 8 P. M.; rising 0.5 ft. per hour. *q*. Max. at 3.30 P. M., 17.35. *r*. Max. at 8 P. M., 22.7. *s*. Max. 5 to 7 A. M., 28.1. *t*. Max. at midnight, 38.6. *u*. Max. at 2 P. M., 36.7. *v*. Mar. 26, 8 A. M., 52.2; Mar. 27, 8 A. M., 53.2, 3 P. M., 53.4 (max.); Mar. 28, 8 A. M., 52.8 and falling. *w*. Previous max. 52.2, in Feb., 1883.

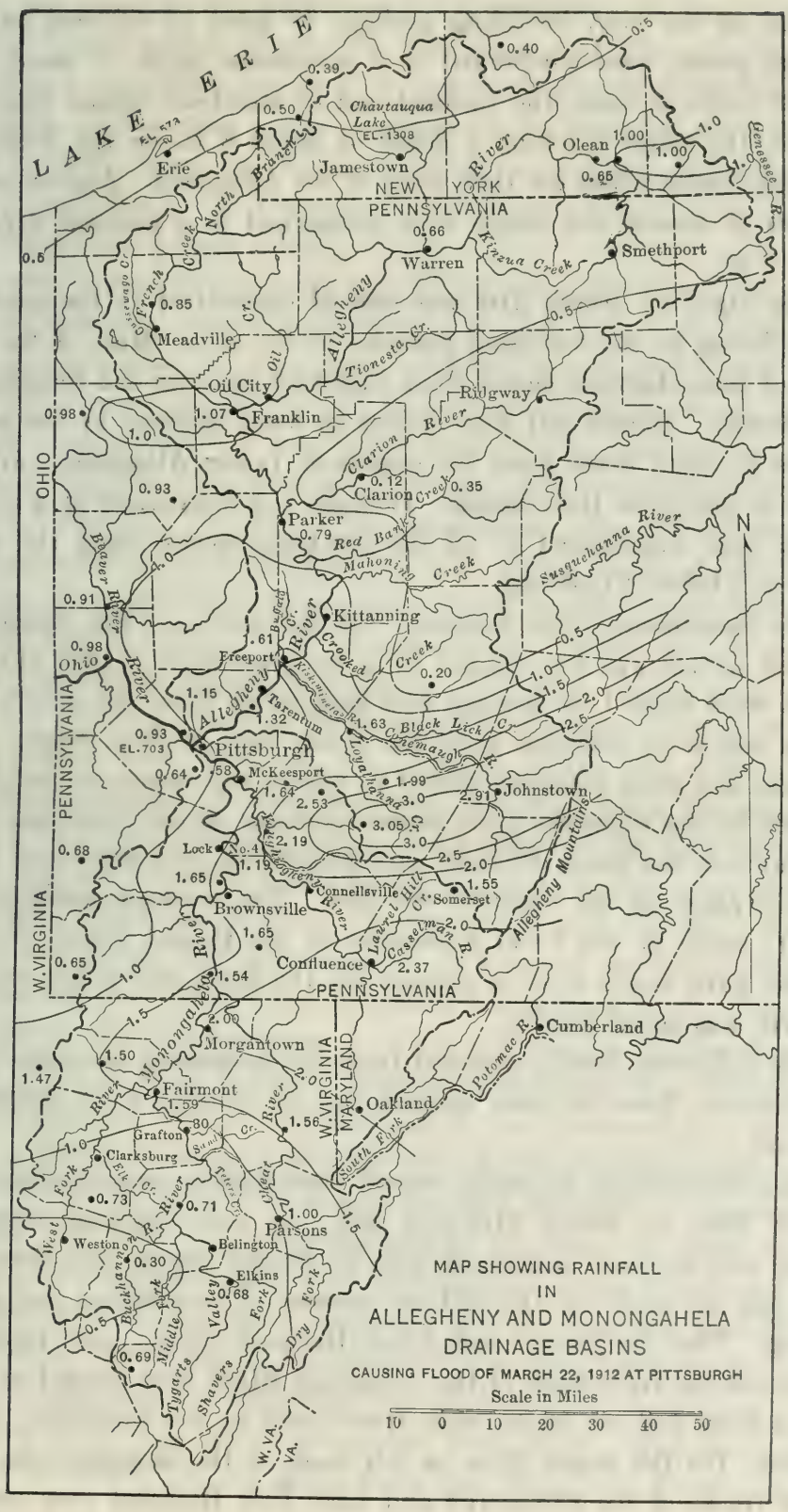

LAKE ERIE
Erie
Jamestown
Chautauqua Lake
EL. 1308
NEW YORK
PENNSYLVANIA
Olean
Warren
Smethport
Meadville
Oil City
Franklin
Ridgway
Clarion
Parker
Kittanning
Erieport
Pittsburgh
McKeesport
Johnstown
Connellsville
Somerset
Brownsville
Confluence
Morgantown
Fairmont
Grafton
Clarksburg
Parsons
Belington
Elkins
Weston
Oakland
Cumberland
OHIO
PENNSYLVANIA
W. VIRGINIA
MARYLAND
Allegheny River
Ohio River
Monongahela River
Kiskiminetas River
Conemaugh R.
Youghiogheny River
Casselman R.
Laurel Hill
Allegheny Mountains
Potomac R.
Genesee R.
Susquehanna River
Clarion River
Oil Creek
French Creek
Conewango Cr.
Tionesta Cr.
Kinzua Creek
Red Bank Creek
Mahoning Creek
Crooked Creek
Black Lick Cr.
Beaver River
Big Beaver Cr.
Tarentum
Lock No. 4
Buffalo Cr.
Brush Cr.
Elk Cr.
Tygarts Valley River
West Fork River
Buckhannon River
Middle Fork
Shavers Fork
Dry Fork
South Fork
W. VA.
VA.
N
MAP SHOWING RAINFALL
IN
ALLEGHENY AND MONONGAHELA
DRAINAGE BASINS
CAUSING FLOOD OF MARCH 22, 1912 AT PITTSBURGH
Scale in Miles
10 0 10 20 30 40 50

FIG. 1.

portions of the city, including perhaps 40 acres of railroad and industrial yards, were submerged to a maximum depth of about 9 ft. Several miles of main-line railroad and of street-car tracks were submerged, the water reaching a depth of about 8 ft. on the Baltimore and Ohio tracks along the right bank of the Allegheny. Approximately 6 miles of streets and alleys were submerged to a greatest depth of about 7 ft.

The night of March 21st was one of discomfort to thousands of people living in the low-lying portions of the city. Most of the residents of these districts moved their furniture upstairs and remained in their houses. Others left their homes in boats manned by the police. Dozens of small boats plied the streets of Lower Allegheny, carrying people to and from their homes. Damage and discomfort of a similar nature were experienced in all the low-lying towns along the rivers above and below Pittsburgh.

The interference with business was not serious. The down-town business section was not invaded, as in the great flood of 1907, although cellars were flooded by seepage and back-water from the sewers, and pumps had to be operated while the rivers were at flood stage. Merchandise and other materials stored in cellars and basements had to be removed to the sidewalks or to higher portions of the buildings. The merchants in the lower districts, as a precaution, had stop-plank floodgates in place on the night of the 21st. The accurate prediction of 28.0 ft., made by the Local Forecaster of the United States Weather Bureau, gave ample time to prepare for the flood, and much valuable property was saved thereby.

The following details gathered from the newspaper accounts of the flood damage, have not been verified, but are believed to be reasonably accurate.

The interruption to traffic caused considerable inconvenience and loss of time on March 21st and 22d. In some sections street-car service was seriously interfered with by flooded tracks, and thousands were late for work on the 22d on account of re-routeing and transferring. The Baltimore and Ohio Railroad tracks and passenger station on the right bank of the Allegheny River were flooded by noon on the 21st, and all trains were transferred to the Smithfield Street Station. On the South Side, or left bank of the Monongahela, some of the tracks of the Pittsburgh and Lake Erie Railroad were under a

few inches of water, but traffic was not interrupted. At West Newton, on the Youghiogheny River, the Baltimore and Ohio tracks were flooded and the trains were run on the tracks of the Pittsburgh and Lake Erie, on the other bank. Portions of the tracks of the Pennsylvania Railroad along the Kiskiminetas River, between Saltsburg and Salina, and along the Conemaugh River, at Sloan's Cut, east of Blairsville, were washed away.

Several thousand men were laid off on account of the high water entering industrial plants and causing their shut-down. At least 2 000 of these employees, it was stated, would be out of work for a week. The Pennsylvania Department of the National Tube Works, in Second Avenue, shut down for a week when water entered the plant at about 2 A. M. on the 22d. The rolling mill at the Continental Department of the same Company also shut down for a week. About 1 700 men are employed in these two departments. The plants of the A. M. Byers Company, at South Sixth Street, and of the Dilworth, Porter Company, at South Fourth Street, closed down on the night of the 21st, on account of the high water backing into the fly-wheel pits. The Carbon Steel Company and the American Steel and Wire Company's plant at 15th Street had to shut down on the morning of the 22d. The Open-Hearth Department of the Jones and Laughlin Steel Company, at South 27th Street, was shut down on the night of the 21st, as it was feared that the furnaces might be flooded, but, as the water did not reach them, work was resumed on the morning of the 22d. At the mills of the Spang, Chalfant Company, at Etna, all movable machinery and material were transferred to points above flood level. This was also done at the Westinghouse Air-Brake Works, at Wilmerding, on Turtle Creek, a small tributary of the Monongahela near Pittsburgh. This plant closed down on the morning of the 21st on account of high water in Turtle Creek, and in the afternoon the water was 20 in. deep on the floor of the works. Men were engaged all night in cleaning up, and work was resumed on the morning of the 22d. At Hays, on the left bank of the Monongahela, a short distance above Pittsburgh, the Monongahela Iron and Steel Company and the National Car Wheel Works were shut down on the night of the 21st, because of flooded furnaces, throwing about 650 men out of work.

Movement of the Flood Down the Ohio River.—The flood crest reached Rochester, Pa., 25 miles down the Ohio from Pittsburgh, at

about 11 A. M. on the 22d, and the streets and cellars in the lower section of the town were flooded. At Steubenville, Ohio, 68 miles below Pittsburgh, the highest stage, 37.0 ft., was reached on the evening of the 22d. Several industrial plants were shut down, and railroad and street-car traffic was interrupted. At Wheeling, W. Va., 90 miles below Pittsburgh, the water rose above the danger line of 36.0 ft. at 1 P. M. on the 22d, and reached the maximum for this flood, 38.6 ft., at about midnight on the 22d. Street-car traffic was interfered with and, on certain lines, was suspended. Basements and cellars in the low districts were flooded, and merchandise had to be removed to the sidewalks and to upper stories. Several mills were obliged to close. The crest reached Parkersburg, W. Va., 183 miles below Pittsburgh, at 4 P. M., on the 24th, when a stage of 37.0 ft., or about 1 ft. above the danger line, was recorded. Little damage is done at this stage at Parkersburg. Cincinnati, Ohio, 468 miles below Pittsburgh, had its highest water, 53.4 ft., or 3.4 ft. above the danger line, at 3 P. M. on March 27th. The flood stage of 50 ft. was reached on the morning of the 25th, and residents in the low-lying districts began to move out of their houses. The water began to overflow some of the railroad tracks on the 26th, when the gauge read 52.2 ft. at 8 A. M., but the service was not interrupted. At Cincinnati a stage of 54 ft. or more prevents trains from entering the Union Central Depot. The river at Cairo, Ill., at the mouth of the Ohio, 967 miles below Pittsburgh, reached its maximum of 54.0 ft. (9 ft. above the danger line) on April 6th. The previous recorded maximum was 52.2 ft., in February, 1883. The river at this point was above the danger line of 45 ft. for 32 days, from March 22d to April 22d, inclusive.

The flood on the Mississippi during April, 1912, was one of the worst ever experienced on that river, and in some sections exceeded all previous records. Many lives were lost, and enormous damage resulted from overflow. The Ohio River was the principal contributor to this rise, although part of the flood-water came from the Lower Missouri and Upper Mississippi. The Arkansas and Red Rivers were not important contributors. Professor H. C. Frankenfield has published* an excellent article recording the history of the conditions preceding this great flood, and the stages of its progress down the river. The data presented in that article bring out the fact that, unquestionably,

* *Engineering News*, April 18th, 1912.

FIG. 2.—WEST RELIANCE STREET, ALLEGHENY, ON THE MORNING OF MARCH 22D, 1912.

FIG. 3.—RIVER AVENUE, ON THE RIGHT BANK OF THE ALLEGHENY RIVER, ON THE MORNING OF MARCH 22D, 1912.

flood stages in the Mississippi are raised by the levees, and that the end has not yet been reached, as far as extreme high stages are concerned, especially in the event of the possible combination of great floods in both the eastern and western tributaries.

Estimated Reduction of the Pittsburgh Crest by Storage Reservoirs.—Before proceeding to a discussion of the reduction of the flood of March 22d, 1912, which could probably have been obtained at Pittsburgh if a system of flood-control reservoirs had been in operation, the following outline of the studies made along these lines by the Flood Commission of Pittsburgh will perhaps be of interest.

The Report of the Flood Commission was issued in April, 1912. It gives a detailed account of the extensive surveys and studies made by the Commission for the purpose of determining the feasibility of preventing floods at Pittsburgh by storage reservoirs on the Allegheny and Monongahela Basins. The forty-three most favorable sites were selected from a large number considered, and detailed estimates were made of the capacities and costs of the respective reservoirs.

These surveys and studies showed that, at certain sites, the maximum storage feasible was greater than needed for flood control. For example, if all forty-three reservoirs were constructed to maximum capacity, the storage in excess of that required for flood-control purposes would amount to 4 357 000 000 cu. ft. on the Allegheny Basin, and 17 937 500 000 cu. ft. on the Monongahela Basin, the total maximum capacity found available being nearly 103 000 000 000 cu. ft.

The capacities of these reservoirs were cut down, therefore, in each case to the amount necessary to control floods from the drainage areas above the respective dams. The total capacity of the twenty-one projects on the Allegheny Basin then amounted to 49 725 800 000 cu. ft., and of the twenty-two on the Monongahela Basin to 30 772 000 000 cu. ft., or a total of 80 497 800 000 cu. ft. These reservoirs would control 8 454 sq. miles, or 73% of the Allegheny drainage area, and 3 379 sq. miles, or 46% of the Monongahela drainage area; or a total of 11 833 sq. miles, or 62% of the total drainage area above Pittsburgh. The total cost of the forty-three projects would be $34 170 800.

An analysis of the effect of these forty-three reservoir projects in reducing floods at Pittsburgh was then made for the eleven principal floods from 1898 to 1908. Prior to 1898, sufficient data for such studies were not available.

This study of the Pittsburgh peak reduction was made graphically by a series of mass-diagrams, in which the estimated flood volumes contributed by the various controlled tributaries were assembled in their calculated positions under the main volume curve representing the Pittsburgh peak, or flood crest, and the reduced peak was obtained by deducting these tributary volumes from the volume of the main peak, which is what would actually take place if the proposed reservoirs on these tributaries were constructed. A diagram of this kind was constructed for each of the eleven floods considered.

By these diagrams it was ascertained that, of the eleven floods studied, all except the highest, that of March, 1907 (35.5. ft.), would have been reduced well below the danger line. The March, 1907, flood would have been reduced to 25.3 ft., and would have been above 22 ft. for 22 hours, as compared with the actual 61 hours. These eleven floods remained above the 22-ft. stage from 34 to 86 hours. The longest flood, that of March, 1905, which reached the gauge height of 29 ft., remained above the 22-ft. stage for 86 hours. This flood would have been reduced to a gauge height of 18.3 ft. It was also found that, with 62% of the total drainage area controlled, a reduction of from 39 to 84% of the peak discharge would have been obtained, or an average of 53 per cent.

Relative Effectiveness of Reservoirs.—It is evident that the amount of flood reduction effected by a given reservoir project is not merely a matter of the magnitude of the flood in the tributary it controls; it is just as much a question of how the contribution of that tributary arrives, with reference to the time of the crest, in the main river.

Inspection of these mass-diagrams, by which the reductions of the Pittsburgh peaks were estimated, showed that certain tributaries are notable and repeated offenders in Pittsburgh floods, that is, their flood-volume curves arrive there at about the same time as the Pittsburgh peak. It is obvious that the most effective storage would be that controlling their flood-waters. Therefore, a study of the relative effectiveness of the various projects was made on this basis, and it was found that, except in the case of the 1907 flood, a comparatively small number of reservoir projects is needed to reduce floods at Pittsburgh to below the danger line; and that certain projects would have but little effect, and, therefore, should be dismissed from further consideration for flood-prevention purposes.

Thus, it was found that if fifteen of these least effective projects were rejected, reducing the cost by about $6 000 000, all the eleven floods considered would still be reduced to below the danger line except that of 1907, which would be reduced to 26.1 ft., or only 0.8 ft. above the point to which it would be reduced by the construction of all forty-three projects.

A further analysis, taking into account the relative cost as well as the relative effectiveness of the respective projects, reduced the number to seventeen. These seventeen selected projects control a drainage area of 8 023 sq. miles, or 69% of the Allegheny Basin, and 2 159 sq. miles, or 29.5% of the Monongahela Basin; or a total of 10 182 sq. miles, or 54% of the total drainage area above Pittsburgh. Thirteen of the projects, with a total capacity of 42 178 500 000 cu. ft., are on the Allegheny Basin, and four, with a total capacity of 17 302 900 000 cu. ft., are on the Monongahela Basin, the total capacity on the two basins being 59 481 400 000 cu. ft. The total cost of the seventeen reservoirs would be $21 672 100, or an average cost of $364 per 1 000 000 cu. ft. of storage capacity.

The locations of these seventeen selected projects, together with the drainage area controlled by each, are shown on Fig. 5.

It was found that, if these seventeen projects had been constructed, the average reduced peak for the eleven floods would have been 18.9 ft.; and that in only the two highest floods, March 15th, 1907 (35.5 ft.), and March 1st, 1902 (32.4 ft.), would the reduction have been less than to the danger line of 22 ft. In the 1902 flood the reduced peak would have been only 0.3 ft. above the danger line, and would have been above that line for such a short time as to be negligible. The reduced peak of the 1907 flood, however, would still have been 6.8 ft. above the danger line, and a certain amount of overflow and damage might still result from occasional great floods, if the flood relief work should be confined to the construction of the seventeen selected reservoirs.

Combination of Reservoirs and Low Wall.—The reduction of the maximum flood to this height, or even to 30.0 ft., however, would greatly reduce the amount of wall necessary to prevent overflow. Investigation showed that the cost of the low wall which could be used in combination with the seventeen selected reservoirs would be so small that it would be far cheaper to construct it than to build the large amount of additional reservoir storage which would be necessary to reduce the occasional great floods below the danger line.

These seventeen selected reservoirs would reduce the flood heights so much that no wall would be required on the Monongahela or Ohio Rivers. It would be necessary to construct a wall only along a few low-lying portions of the Allegheny. This low wall, in combination with the reduction in flood heights which would be obtained by reservoir storage, would prevent overflow by a 40-ft. flood at Pittsburgh. It would cost about $670 000. The total cost, therefore, of the wall and reservoirs would be $22 342 100.

If certain revisions in the river channels were made, the total value of the land reclaimed along the river front would amount to about $2 307 000. The total net cost of the low wall and the seventeen selected reservoirs combined, therefore, would be $20 035 100.

Reduction of the Flood of March 22d, 1912.—The flood of March 22d, 1912, is the first of any importance that has occurred since gauging stations have been in operation on all the tributaries controlled by the seventeen selected reservoirs referred to. The accompanying graphical estimate (Fig. 6) of the probable reduction in the Pittsburgh peak which could have been obtained had the system of reservoirs recommended by the Flood Commission been in operation is, therefore, of peculiar interest, as confirming the claims made by the Commission and showing the conservative character of the assumptions made necessary by incomplete data in certain portions of the above described flood-reduction studies.

The peak reduction diagram, Fig. 6, is similar to the diagrams used in the studies made by the Flood Commission. The discharges at Pittsburgh were obtained by gauge heights furnished by the U. S. Weather Bureau, and an approximate discharge curve for the Ohio River at Pittsburgh was based on measurements by the U. S. Engineers. The flood-volume curves of the tributaries controlled by the seventeen selected projects are, with two exceptions, based on actual stream-flow measurements. The curve representing the flood discharge of the Allegheny River at the site of the lower of the three Allegheny projects, a short distance above Oil City, is based on discharge measurements at Red House, N. Y.; the Loyalhanna Creek curve is based on the records of Black Lick Creek and a study of rainfall data, the Loyalhanna gauge having unfortunately been out of commission during this flood.

These tributary flood-volume curves, representing the flood-water which would have been held back by the proposed reservoirs, are plotted under the Pittsburgh peak in the positions in which it is estimated

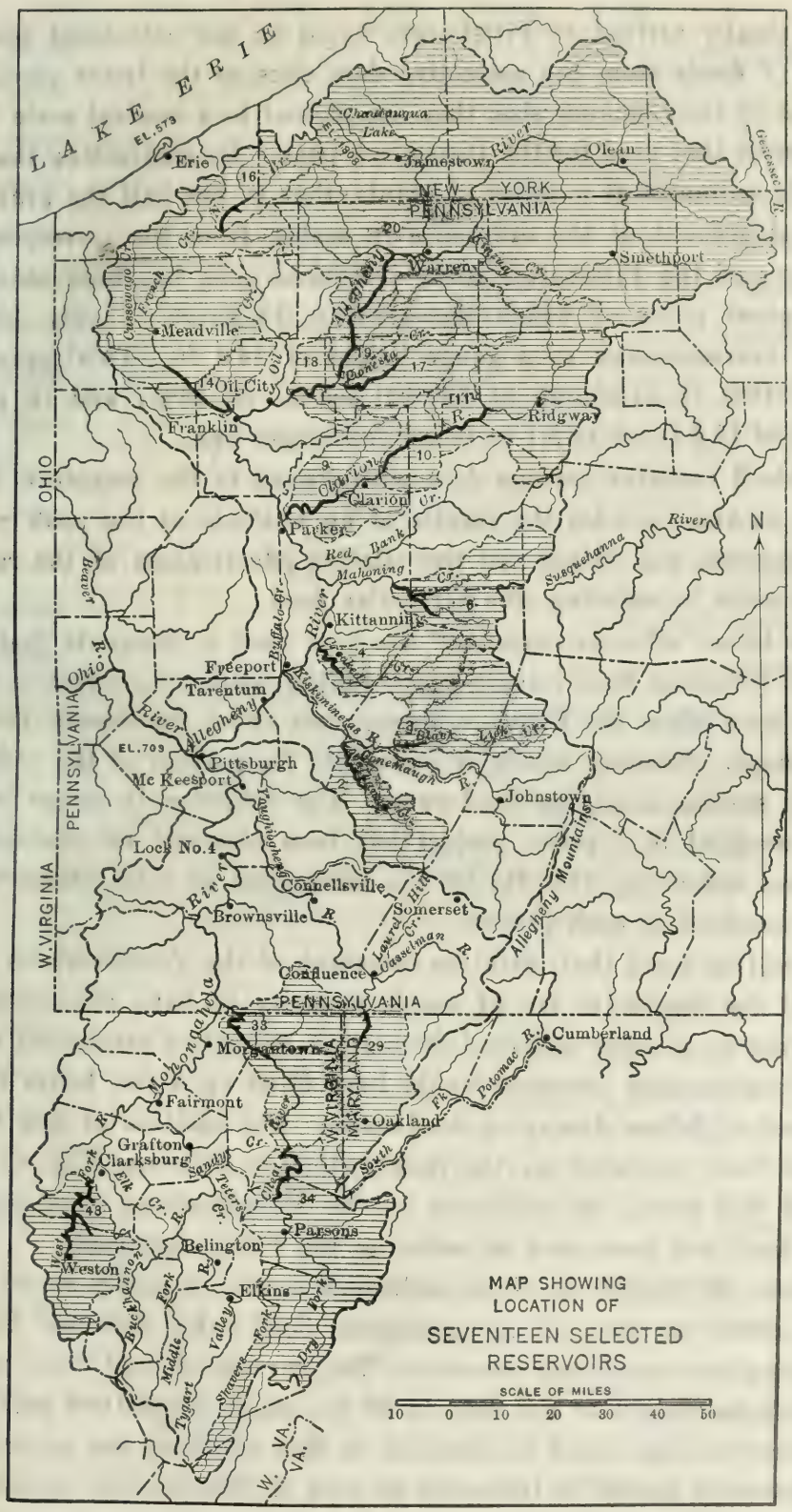

LAKE ERIE
EL. 573
Erie
Chautauqua Lake
Jamestown
Olean
NEW YORK
PENNSYLVANIA
Warren
Smethport
Meadville
Oil City
Franklin
Ridgway
Clarion
Parker
Kittanning
Freeport
Tarentum
Pittsburgh
EL. 709
Mc Keesport
Johnstown
Lock No. 4
Connellsville
Somerset
Brownsville
Confluence
Cumberland
Morgantown
Fairmont
Oakland
Grafton
Clarksburg
Parsons
Belington
Weston
Elkins
OHIO
PENNSYLVANIA
W. VIRGINIA
MARYLAND
Ohio R.
Allegheny River
Monongahela River
Youghiogheny R.
Kiskiminetas R.
Conemaugh R.
Susquehanna River
Potomac R.
Allegheny Mountains
Red Bank
Mahoning Cr.
Clarion R.
Little Toby Cr.
Laurel Hill
Casselman R.
Middle Fork
Tygart Valley
Shavers Fork
Cheat R.
W. VA.
VA.
N
MAP SHOWING
LOCATION OF
SEVENTEEN SELECTED
RESERVOIRS
SCALE OF MILES
10 0 10 20 30 40 50

FIG. 5.

they actually arrived at Pittsburgh, based on the calculated time of travel of floods from the respective dam sites to the latter place. On account of their smaller size, they are plotted to a vertical scale twice as great as that used for the Pittsburgh peak. By subtracting the sum of their ordinates at a series of points, that is, one-half the graphical sum, on account of the difference in scales, from the corresponding ordinates of the Pittsburgh peak, a reduced peak has been obtained, the highest point of which represents a discharge of only 168 000 sec-ft., corresponding to a gauge height of 16.9 ft. This represents a reduction in discharge of 132 800 sec-ft., or 43%; and in gauge height, of 11.2 ft., or to 5.1 ft. below the danger line.

Table 3 contains various data with regard to the seventeen reservoirs. It also includes the results of an analysis of the peak reduction diagram, and brings out the relative effectiveness of the respective projects in reducing this particular flood.

The term, "effective capacity," has been used to designate that part of each tributary flood-volume curve falling within the effective zone, or the time when the Pittsburgh crest was above the danger line, or 22-ft. stage. In other words, it represents the portion of the capacity used in storing damaging flood-water. The reduction in gauge height to be credited to a given project has been obtained by multiplying the total reduction, 11.2 ft., by the percentage of total effective capacity credited to each project.

It will be noted that, with the exception of the Youghiogheny project, all the reservoirs are of ample capacity to have controlled the damaging flood-water delivered from their respective catchment areas. The Youghiogheny reservoir would have filled up a few hours before it ceased to deliver damaging flood-water. The position of this filling up has been indicated on the peak reduction diagram, Fig. 6; and, beyond this point, the ordinates of the Youghiogheny flood volume curve have not been used in reducing the Pittsburgh peak.

About 293 000 000 cu. ft. of additional storage capacity would have been needed to store all the damaging flood-water delivered by the Youghiogheny above the reservoir. The surveys showed that, by increasing the height of the dam to 96 ft., about 984 000 000 cu. ft. of additional storage could be obtained at this site, and the capacity of this reservoir should be increased by part of this amount, at least, in order to insure complete control of all floods from its catchment area.

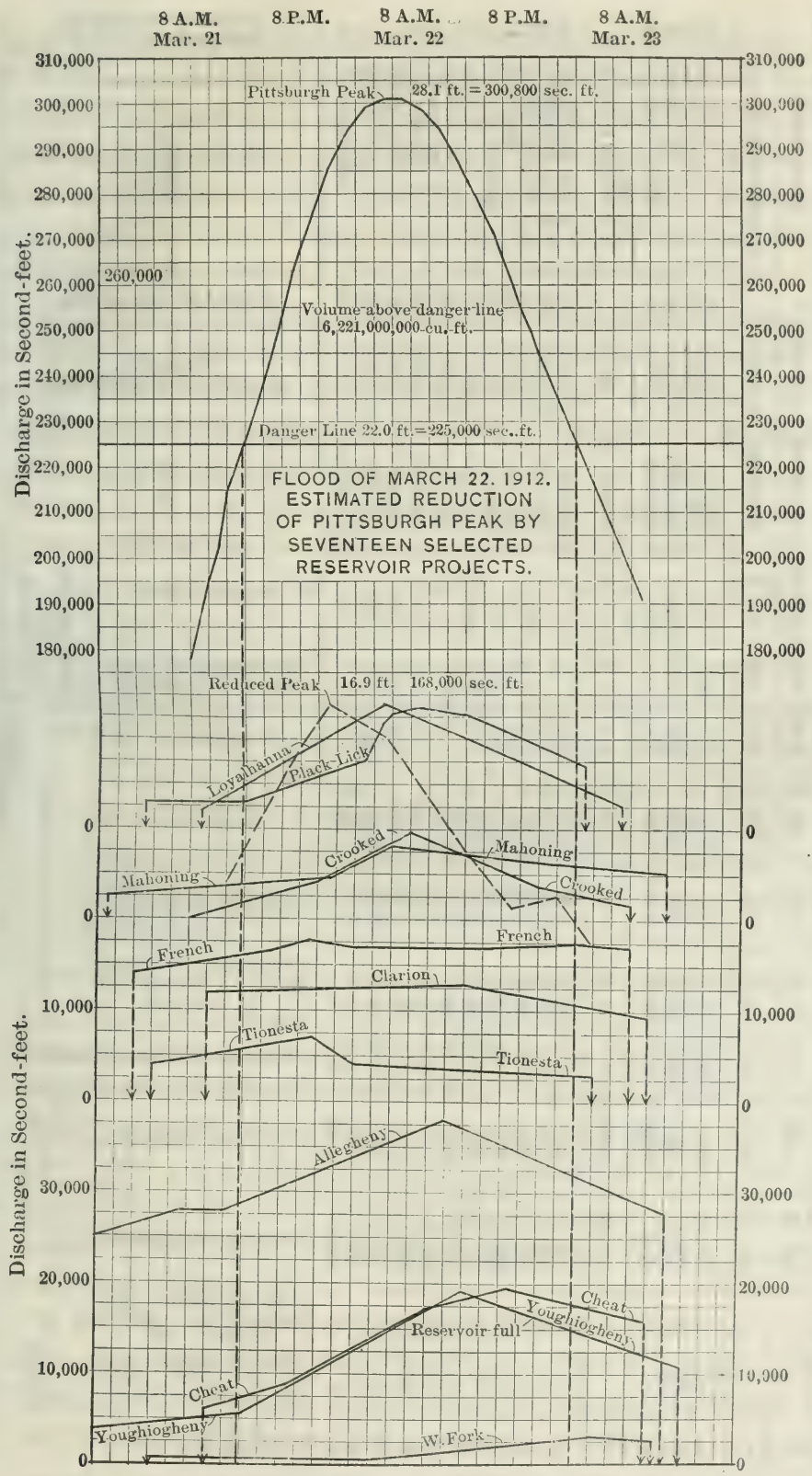

8 A.M. Mar. 21
8 P.M.
8 A.M. Mar. 22
8 P.M.
8 A.M. Mar. 23
Discharge in Second-feet.
Pittsburgh Peak 28.1 ft. = 300,800 sec. ft.
260,000
Volume above danger line 6,221,000,000 cu. ft.
Danger Line 22.0 ft. = 225,000 sec. ft.
FLOOD OF MARCH 22. 1912. ESTIMATED REDUCTION OF PITTSBURGH PEAK BY SEVENTEEN SELECTED RESERVOIR PROJECTS.
Reduced Peak 16.9 ft. 168,000 sec. ft.
Loyalhanna
Black Lick
Crooked
Mahoning
Mahoning
Crooked
French
French
Clarion
Tionesta
Tionesta
Allegheny
Cheat
Youghiogheny
Reservoir full
Cheat
Youghiogheny
W. Fork

FIG. 6.

TABLE 3.—REDUCTION OF FLOOD OF MARCH 22D, 1912, AT PITTSBURGH BY STORAGE RESERVOIRS.

Number indicating reservoir on Fig. 6.	Project.	Height of dam, in feet.	Capacity, in millions of cubic feet.	Cost.	Drainage Area Controlled.		Percentage of total cost.	Effective Capacity.			Reduction in gauge height.	Cost per Million Cubic Feet of Capacity.		Ranked According to:		
					Square miles.	Percentage of total above Pittsburgh.		Millions of cubic feet.	Percentage of total capacity of project.	Percentage of total effective capacity of all projects.		Of total capacity.	Of effective capacity.	Cost per million cubic feet of total capacity (lowest = No. 1).	Cost per million cubic feet of effective capacity (lowest = No. 1).	Effectiveness. (Greatest = No 1.)
2	Loyalhanna	122	4 112.5	$1 222 060	277	2.4	5.6	1 242.0	30.2	7.52	0.84	$297	$984	7	3	4
3	Black Lick	63	1 454.7	720 500	414	3.6	3.3	1 188.0	81.7	7.17	0.80	495	607	13	1	6
4	Crooked	94	3 255.7	893 700	287	2.4	4.1	727.2	22.3	4.40	0.49	274	1 229	5	6	11
6	Mahoning	143	2 367.8	1 089 400	335	2.9	5.0	828.0	35.0	5.00	0.56	460	1 315	12	8	10
9	Clarion No. 1	142	5 067.1	1 309 900	1 212	10.5	6.1	691.2	13.6	4.18	0.47	258	1 895	4	14	12
10	Clarion No. 3	128	4 886.6	1 028 100	.....	....	4.8	666.0	13.6	4.03	0.45	210	1 544	1	11	13
11	Clarion No. 4	70	1 537.9	451 800	.....	....	2.1	208.8	13.6	1.26	0.14	294	2 164	6	15	16
14	French	75	3 323.1	2 388 100	1 009*	8.6	11.0	1 836.0	55.2	11.10	1.24	719	1 301	17	7	2
16	No. Br. French	67	2 125.7	722 800	217	1.9	3.3	396.0	18.6	2.40	0.27	340	1 825	9	13	15
17	Tionesta	103	3 629.6	1 362 600	477	4.1	6.3	601.2	16.6	3.64	0.41	375	2 266	10	16	14
18	Allegheny No. 1	63	2 876.3	1 219 900	3 795†	32.8	5.7	1 231.2	43.0	7.45	0.83	424	991	11	4	5
19	Allegheny No. 2	66	4 877.9	3 057 000	.....	....	14.1	2 095.2	43.0	12.67	1.42	627	1 459	15	9	1
20	Allegheny No. 3	54	2 663.6	1 386 000	.....	....	6.4	1 144.8	43.0	6.92	0.78	520	1 211	14	5	7
	Total Allegheny	.....	42 178.5	$16 851 800	8 023	69.2	77.8	12 855.6	30.4	77.74	8.70	$400	$1 311	..	..	..
29	Youghiogheny	76	1 547.1	$999 300	394	5.4	4.6	1 547.1	100.0	9.36	1.05	$646	$646	16	2	3
33	Cheat No. 1	113	5 737.4	1 287 900	1 399	19.1	5.9	849.6	14.8	5.14	0.58	224	1 516	2	10	9
34	Cheat No. 2	136	7 294.1	1 721 900	.....	....	8.0	1 080.0	14.8	6.52	0.73	236	1 594	3	12	8
43	W. Fork	66	2 724.3	811 200	366	5.0	3.7	205.2	7.5	1.24	0.14	298	3 953	8	17	17
	Total Monongahela	.....	17 302.9	$4 820 300	2 159	29.5	22.2	3 681.9	21.2	22.26	2.50	$279	$1 309	..	..	..
	Grand total	.....	59 481.4	$21 672 100	10 182	53.8	100.0	16 537.5	27.8	100.00	11.20	$364	$1 310	..	..	..

NOTE.—Total drainage area of Allegheny River is 11 580 sq. miles; of Monongahela River is 7 340 sq. miles.

* Excluding area controlled by No. Br. French project. † Excluding area controlled by Tionesta project.

It should be noted, however, that before the discharge of about 17 000 sec-ft., which the Youghiogheny was delivering at the time the reservoir would have filled up, could have been reached in the stream below the dam, enough water would have been stored in the reservoir to give a head of about 2.5 ft. on the spillway. With a surface area of 940 acres, this would have given an additional storage of about 102 000 000 cu. ft.

The costs per 1 000 000 cu. ft. of total and of effective capacity are also shown in Table 3. In the last three columns, the projects are ranked according to these costs, the lowest being No. 1; and, according to effectiveness, the project with the greatest reduction in gauge height to its credit is ranked as No. 1.

Evidently, it does not follow that, because a reservoir has a low cost per 1 000 000 cu. ft. of storage, it is the cheapest to build for flood-control purposes. The water which it controls may be that of a tributary which rarely, if ever, delivers that flood-water at Pittsburgh during the critical period of damaging flood height. Whereas, another reservoir, of high cost per unit of storage, may control a stream which is invariably an offender whenever its drainage area receives flood rainfall. A number of favorable reservoir sites having been selected and surveyed, and estimates of the capacities and costs of the respective projects having been determined, if only a certain number of them are to be built, the ultimate criterion for their selection for flood-control purposes is not the lowest cost, nor even the greatest effectiveness; it is both. In other words, the final selection should be based on the lowest cost per unit of effectiveness.

Thus, in Table 3, it will be noted that, in this particular flood, the Clarion No. 3 project ranked 13 in effectiveness and 11 in cost per unit of effectiveness, although it ranked 1 in cost per unit of total capacity. The Youghiogheny project, on the other hand, although it ranked 16 in cost per unit of total capacity, being exceeded only by the French Creek project, ranked 3 in effectiveness and 2 in cost per unit of effectiveness.

It is obvious, of course, that such a selection of reservoirs cannot be made from a study of one flood, for no two floods are alike in their origin. The rainfall varies widely in amount, in distribution, and in relative time of arrival. Moreover, even with the same rainfall, the run-off from a given part of a drainage area, being affected by

many conditions, may vary widely. It is necessary, therefore, that the volume and distribution of storage shall be adequate to control all possible conditions and combinations of run-off.

It was in the foregoing manner, after making a similar analysis of the action of the forty-three reservoirs in the eleven principal floods from 1898 to 1908, that the seventeen reservoir projects recommended by the Flood Commission of Pittsburgh were selected. As previously stated, it is estimated by the Commission's engineers that, with these seventeen reservoirs, all floods at Pittsburgh, including a possible future 40-ft. flood, would be reduced to a point where overflow and flood damage could be prevented by a low wall along a few low-lying portions of the Allegheny River; and that the total net cost of these reservoirs and the wall, taking into account the value of the land reclaimed, would be only about $20 000 000, or less than twice the direct losses from flood damage sustained during the past ten years at Pittsburgh alone. Moreover, the flood relief would not be confined to Pittsburgh, but would be extended along the rivers and their principal tributaries above that city, and down the Ohio River for many miles below it. For example, at Wheeling, W. Va., 90 miles below Pittsburgh, the greatest recorded flood, that of 1884, 53.1 ft., could have been reduced by 13 ft.; and the next greatest, that of March, 1907, 50.1 ft., by 14.5 ft.

Proceeding farther down the Ohio, it is evident that, as other important tributaries enter, this storage would become a decreasing factor in reducing the Ohio floods. In so far as such floods originate above Pittsburgh, the storage would be effective, but complete control of floods at these lower points would necessitate the construction of additional storage on the tributaries entering the Ohio between Pittsburgh and the point where flood relief is desired. A similar solution of the problem on the other tributaries of the Ohio would extend the flood relief throughout its entire valley.

Above Pittsburgh, the Allegheny River, from its mouth to Oil City —a distance of about 134 miles—would be relieved from frequent disastrous floods. Flood levels on the Kiskiminetas River, for a distance of 41 miles above its mouth, would also be considerably lowered. On the Monongahela River, floods would be greatly reduced from Fairmont, W. Va., to Pittsburgh, a distance of 127 miles. On the West Fork River, they would be controlled for a distance of 38 miles above the mouth, and, on the Youghiogheny River, they would be lowered considerably for a distance of 83 miles above its mouth.

The system of storage reservoirs could be operated primarily for flood prevention during the flood season, and for increasing the low-water flow during the low-water season. A study of the flood records shows that 80% of the floods has occurred in the months from November to March, inclusive; and that the 20%, occurring in the period from April to October, inclusive, has been of relatively less severity, and could have been prevented from doing damage by the low wall which has been proposed in combination with the reservoirs.

With the amount of flood-water which, by this method, could be safely retained in these reservoirs until needed in dry weather, combined with the surplus spring and early summer flow that could be gathered, a notable improvement in the low-water flow of the main rivers and their tributaries could be effected. The low-water flow of the Allegheny at Pittsburgh could be increased to about 3.5 times its minimum throughout the entire dry-weather season; that of the Monongahela to about 6 times its minimum; that of the Kiskiminetas River to about 6 times its minimum; and that of the Youghiogheny River to about 10 times its minimum. These increases of low-water flow are those which would obtain with only the seventeen selected projects constructed, and would, of course, extend down the Ohio. At Wheeling, the low-water discharge could be maintained at 3 times the present minimum, corresponding to an increase in stage of 2.3 ft.

The extent of these improvements in low-water flow will be appreciated when it is realized that 267 miles of main rivers and 386 miles of tributaries, or a total of 653 miles of stream channels above Pittsburgh, would have their low-water discharge considerably increased and made uniform during dry weather. If all the forty-three projects referred to were constructed, there would be, of course, a considerably greater increase of low-water flow, which would extend over 393 miles of main rivers and 570 miles of tributaries, or a total of 963 miles of stream channels above Pittsburgh.

The benefits to navigation, sanitation, water supply, and water-power, which would result from such an improvement in stream regimen would naturally be very considerable. They form a strong additional argument in favor of the construction of storage reservoirs for purposes of river regulation and flood control, and broaden the problem to one demanding State and National consideration.

DISCUSSION

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter.)—The writer is deeply interested in this paper, because the general scheme for flood control therein considered was, in his younger days, a pleasing day-dream which haunted him for years. Long experience in this line of thought, however, has brought a realization of many practical difficulties which cannot be easily adjusted. The author says:

"The system of storage reservoirs could be operated primarily for flood prevention during the flood season, and for increasing the low-water flow during the low-water season."

This is perfectly feasible, and true in every respect, provided every private or quasi-private interest is prevented from interfering with free and untrammelled operation in the interest of flood control purely.

The author also states:

"The benefits to navigation, sanitation, water supply, and water power, which would result from such an improvement in stream regimen would naturally be very considerable."

This is exactly where the fundamental difficulties of the whole scheme come in. Experience everywhere shows that it is almost impossible to reconcile private interests and flood-control interests in one and the same scheme. The conflict is irrepressible, and, in a majority of cases, the combined scheme is utterly impracticable. A single instance is sufficient to show the true nature of the irrepressible conflict. The same reasoning will apply to any one of the seventeen reservoirs proposed.

Complete flood control requires that all the storage reservoirs shall be practically empty just before the expected peak flood arrives, say, in February and March of each year. This will furnish the desired storage room for the great rush of flood-waters, and everything will work satisfactorily. On the contrary, how does this requirement affect private interests? Water-supply and water-power interests both demand that the reservoir shall be filled as early in the wet season as possible so as to be absolutely sure of a full reservoir before that season is over. This means that the expected great "peak flood," which comes in February and March, will rush down the river and into a full reservoir, pass over the crest of the dam unrestrained, and continue down stream just exactly as it did before the dam was built. As a result, the flood height in the lower river would be the same as before the reservoir system was built, if not greater, inasmuch as the proposed improvements are supposed to raise the low-water plane of the river to some extent.

From this it is clear that, as a rule, flood-control schemes cannot be combined with such schemes as water supply, water power, irriga-

Mr. Le Conte.

tion, etc., on account of conflicting interests which are almost insurmountable. This is the principal reason why this truly fascinating problem has remained dormant for sixty years or more, and has never received a practical solution.

The writer ventures to submit a few mild suggestions, which, of course, pass for what they are worth. Where the flood-danger period does not extend over 2 or 3 months each year, the storage reservoir waters could be used for power or water-supply purposes during the remaining 9 or 10 months. During the 2 or 3 months when the storage reservoirs must be kept practically empty—in anticipation of the great peak flood—all the power plants and water-supply plants must necessarily be kept going with auxiliary steam plants erected for the special purpose. The long transmission lines in California generally have auxiliary steam plants in the cities where they sell their power and light. These auxiliary plants are started up whenever there is a breakdown on the main transmission lines; therefore, a stoppage of 2 or 3 months at the main power-house is not vital. Likewise, in the case of a water-supply company: if the natural flow of the stream above the reservoir site, in February and March, is sufficient for water-supply purposes, all well and good, and no steps need be taken for an auxiliary supply; but, if it be short of requirements, a small auxiliary steam pumping plant could be located just below the dam site, the pump wells being fed by underground seepage from the reservoir above. The capacity of this plant need be only sufficient to cover the deficiency.

Mr. Copeland.

William R. Copeland, Assoc. Am. Soc. C. E. (by letter).—Mr. Grant evidently prepared his paper with the idea of proving that the construction of storage reservoirs on the water-shed of the Allegheny River will protect the residents of Pittsburgh and vicinity from damage by flood. He seems to have overlooked an important matter in this connection, and that is the question of the effect which may be produced on the Allegheny by the storage of acid waters from coal mines in such reservoirs.

Consider, for example, the proposed reservoir on the Loyalhanna. This river, rising on the western slope of the Allegheny Mountains, flows northwestward for about 30 miles until it joins the Conemaugh at Saltsburgh, forming the Kiskiminetas. From its source to Latrobe, it lies in a rather broad, open valley. Between Latrobe and Saltsburgh, however, it winds through a valley, so narrow and deep in many places that it becomes a gorge.

This valley is crossed at four or five places by dikes of stone which form natural dams, and they have been raised in height artificially for the purpose of storing water for mill powers. The pools thus formed—each a mile or more in length—serve as catch-basins for the mine drainage entering the stream from each side.

Mr. Copeland.

This mine water has several marked characteristics, one of which is that its specific gravity is greater than that of ordinary surface water. Therefore, the drainage drops to the bottom of the river, collecting, of course, in the pools above the dikes. Another bad feature of mine drainage is the free acid and acid salts which it contains.

Chemical analyses have shown that water flowing from coal mines in this region carries from 100 to 500 parts per million of free acid. Dilution and reaction with the alkaline carbonates carried by the surface waters tend to reduce the acidity, but, nevertheless, a sample of water taken from the bottom of a pool in the Loyalhanna in September, 1899, contained 200 parts per million of free acid—and the sample was taken just after a considerable flood had passed down the stream.

July, August, and the first half of September had been very dry, but about September 15th a thunder-storm having the characteristics of a cloudburst broke upon the upper water-shed. So great was the rainfall that the run-off raised the river more than 2 ft., creating a current in the Loyalhanna which swept the immense volumes of acid mine drainage stored in the pools into the Kiskiminetas, and eventually into the Allegheny.

Millers and farmers have complained for years that the mine waters of the Loyalhanna eat up their iron water-wheels, and poison stock which drink from the stream. One man told the writer that he had lost a 1-in. iron crow-bar through the ice on the river during the preceding winter, and that by spring the bar had been eaten in two.

The head-waters of the Loyalhanna contain fish, frogs, lilies, and all the natural aquatic life of the region; but below the entrance of the first coal-mine drains, at the outskirts of Latrobe, not a fish nor a weed can be found in the water.

When the mine water—swept into the Kiskiminetas by that September flood—reached its month, the acid burned the legs of Italian laborers working on a bridge pier, and drove them out of the river. Passing into the Allegheny the sulphur water killed tons of fish in its run of 40 miles to Pittsburgh, and the rotting bodies created such foul conditions that the Superintendent of the Pittsburgh water supply at once started an investigation regarding the cause of the death of the fish.

Smaller floods of this character are of common occurrence in the Allegheny, and are becoming more common annually. The volume of mine water is increasing, too, as new coal mines are being opened yearly, the workings in others are being extended, and last, but by no means least, the drainage from all former openings continues to pour unceasingly into the watercourses.

Mr. Copeland.

At present the dikes and dams are so low that the Loyalhanna is flushed out several times a year, but what will happen if a great reservoir is formed by throwing a dam across the stream near its mouth, as indicated on the plan, Fig. 5. Such a structure will surely store the acid mine drainage, and, if any exceptional flood, from a cloudburst or other severe storm, sweeps the water out of the reservoir, far greater volumes of stronger acid water will be poured into the Allegheny than has ever entered it at one time before.

When this happens, the water consumers in all the towns on the river, and the men in charge of the water purification works at Pittsburgh and elsewhere, may well take heed lest the mine drainage destroy their boilers and water mains, and even close the water-works plants temporarily.

The location of the dam on the Loyalhanna has doubtless been chosen in order to get a maximum amount of storage and to decrease the danger of flood from this region to a minimum; but, in view of the presence of mine water on the lower water-shed, is it advisable to locate the dam at the proposed site? Would it not be better to build the dam above Latrobe, for instance, at Ligonier? Part of the town, a railroad, and some highways would doubtless be wiped out, and the volume of water stored would be reduced. Some persons might argue, further, that the process of holding back the alkaline surface water at Ligonier would cause a more concentrated sulphur water to flow into the Allegheny. This effect could be easily remedied by opening gates at the dam, and really improve present conditions by keeping a larger and more uniform flow through the lower valley at all seasons.

If the dam is ever built at the site proposed in Fig. 5, channels should be cut through all the present dikes to drain the heavy acid water from the lower parts of the pools; and blow-off gates should be placed in the bottom of the dam through which a constant discharge of water will be maintained for the purpose of preventing the acid mine drainage from destroying the gates or even injuring the structure itself, as well as the city mains and plants on the lower rivers.

Mr. Knowles calls attention to the fact that the Flood Commission considered the question of the action of acid mine waters in storage reservoirs, and decided that there was no danger from this source. The speaker desires to add a word of caution, because there are two factors which, from the nature of the acid mine water and well-known hydraulic laws, contradict the reasons suggested by Mr. Knowles.

Mine water is heavier than surface water, and can be made to mix with it only by prolonged agitation. The theory that because mine water is brought into the presence of great volumes of surface water the acidity will be neutralized by dilution is disproved by the evidence

Mr. Copeland.

presented by such rivers as the Loyalhanna and Youghiogheny. The acid mine drainage entering the latter from the coal mines of the Connellsville region gathers in pools at the bottom of the river as far away as McKeesport. This fact can be verified by the manager of the water-softening plant at that city.

Moreover, all persons who have to deal with solutions—especially with those of metallic salts—find that constant agitation for hours is required to make the heavy liquors mix with the dilution waters, and that this agitation must be kept up constantly in order to prevent the two kinds of waters from separating into layers.

Four or five years ago the writer traced the acid mine water flowing out of the Kiskiminetas down the left bank of the Allegheny River as far as Tarentum—a distance of about 10 miles. The condition was all the more noticeable because an island, which lies in the river about ½ mile above Tarentum, divided the stream into a streak of clear strongly acid water on its left shore and muddy and strongly alkaline waters on its right shore. This fact was not confined to the writer's observation, but was confirmed by the ferryman who lived on the left bank of the river, at the foot of the island, and by the officials of the Pennsylvania Salt Company's works at Natrona. These gentlemen, whose plant was under suspicion at the time as the source of the acid water in the river, had made tests on many samples of the river water, and found that it was acid for a large part of the year along the left bank and alkaline on the right.

Moreover, the surface water carried into the Loyalhanna by spring rains will probably be several degrees warmer than the water which has been lying at the bottom of the reservoir all winter. This difference in temperature will tend to make the surface water run over the top of the mine drainage lying in the deep pools on the floor of the basin.

Finally, it is a matter of common observation that a sudden inrush of flood-waters tends to push the water in the reservoirs out bodily, instead of mingling with it. Therefore, the promoters of these storage reservoirs in coal-mine districts are warned that Nature is not likely to destroy the acidity by dilution, and that the builders of dams must provide artificial means for making surface water mix with the mine waste, otherwise the building of dams will be dangerous to the inhabitants of the lower water-sheds.

The writer desires to suggest a second time that the pools in the bottoms of these reservoirs be filled with sand and gravel or drained, and that gates, left open in the bottoms of the dams, shall maintain a constant discharge of the heavy liquors collecting along the beds of the reservoirs.

Jean de Pulligny, M. Am. Soc. C. E.—The speaker knows very little about the Ohio River, as he only saw it once, but can say a few words about the very severe floods which have to be controlled in Paris, where they caused great damage three years ago. At the present time the speaker understands that the waters are rising again very rapidly, and the people are anxious to know whether they will have to undergo the same severe experience. Mr. de Pulligny.

One of the solutions proposed has been the building of dams up stream. The Seine has several affluents, and the rising of the flood at Paris depends largely on the way in which the various rivers above send down their flow. If the floods arrive at Paris simultaneously, the rise there is very high; if they pass one after the other, the rise is much less, so that keeping one flood waiting a few hours and allowing it to escape afterward might be sufficient to prevent any great rise at Paris.

Besides the construction of reservoirs up stream, other measures have been proposed, and, generally speaking, all those which allow the water to flow freely between the city embankments and below the city may help, to a certain extent, to lower the crest of the flood.

It has been proposed to build, around the city, a deviation canal which would receive the flow of the Marne (an affluent of the Seine) above Paris, and carry it below the city. For helping the flow of waters below Paris, it has been proposed to construct an underground canal from Bougival to Poissy, which would cut straight across the several loops formed by the river below the city. All these works and many others would surely be useful, but, unfortunately, their cost would be very large.

J. Waldo Smith, M. Am. Soc. C. E.—The speaker is not familiar with the Ohio River, or with the circumstances or conditions which control the flow of the Allegheny in the Pittsburgh District, but if these streams are like those with which he is familiar in New York State, New England, and New Jersey, he is convinced that there has been no great flood, but that at some time there will be a greater one, and that there has occurred no great drought, but that at some time there will be a drier one. Mr. Smith.

To control these floods, and to utilize their waters by storage, is an extremely difficult problem. Floods in the rivers with which the speaker is familiar do not occur by rule; they are likely to occur in any month of the year. He has known of floods in July, in August, in September, in October, in January, March, April, May, and June, and, no doubt, a careful comparison of the records would show that floods had occurred in the other months of the year as well, and severe floods too. In order, then, to utilize the run-off waters for the development of an average power or for augmenting the low-season flow

Mr. Smith. and, at the same time, furnish adequate protection against floods, is a difficult problem.

This discussion is a general one, for, as already stated, the speaker is not familiar with the problems of the Allegheny River, and it may be that floods in that region usually occur in certain seasons only.

Mr. Knowles. MORRIS KNOWLES, M. AM. SOC. C. E.—To the speaker and to others who have studied the conditions in Pittsburgh, this paper is exceedingly interesting. It will be recalled that some criticisms of the Flood Commission's Report have been based on the fact that it was founded on incomplete data regarding rainfall, run-off, and river gaugings; yet this paper is a study, using actual rainfall measurements and stream gaugings on every one of the streams on which the Flood Commission recommended reservoirs. The close correspondence of the results to those calculated on necessarily incomplete data for the Report proves the reasonableness of the Flood Commission's assumptions and justifies its belief that the seventeen recommended projects would prove effective in controlling floods.

Another interesting feature is the light thrown on the one flood which would not have been lower than the damage height, even with the seventeen reservoirs in operation, namely, the flood of 1907. The main causes at the time of that flood were the Kiskiminetas and the Youghiogheny, just as in the flood of 1912, although not exactly to the same degree. This study confirms the view that even these two offending streams could have been controlled (when we consider their time and intensity effect acting with all others) so that conditions would have been materially better.

A word as to the recurrence of floods, and as to the reasonableness of considering any project, not only for this purpose, but for other and correlated purposes: During the past 38 years, about 75% of the floods at Pittsburgh have occurred between December 1st and April 1st; the remaining 25% have been scattered through the other months, and, with one or two exceptions, have been small and of short duration. Of this 25% not one would have topped the flood wall which the Flood Commission recommended to supplement the seventeen reservoirs. The reservoirs, therefore, would have been necessary for flood protection only during 4 months of the year, and could have been operated for other purposes during the remaining 8 months. It was this which led the Pennsylvania Water Supply Commission to incorporate such restrictions as the following in two power company charters that have been granted since the Flood Commission made its report:

"That the requirements of the Corps of Engineers, United States Army, in charge of the Allegheny River, as to the minimum stream discharge must be embodied in any plan for using the water stored, as well as the rights of lower riparian owners to have available at all

times at least the minimum stream flow, as determined by the Water Supply Commission of Pennsylvania, must be protected. Mr. Knowles.

"That the operation of reservoirs, in so far as the control of floods and the maintenance of low-water flow is concerned, shall be subject to the direction and jurisdiction of the Water Supply Commission of Pennsylvania."

There has been some discussion with regard to the effect on acid conditions. The speaker had hoped that the report of the Flood Commission had made it evident that in this matter, also, there was great benefit to be obtained by this plan. All but four of the proposed reservoirs are located on portions of streams which are not now acid. It is true that some others may become acid, but the location of the reservoirs, near the head-waters, and above most of the mine developments, will restrict this to a minimum. Moreover, the effect at the low-pool dams is no criterion whatever of what will happen in a very large basin with a comparatively high dam. There a sudden rise will not flush out the acid which has accumulated in the bottom, but will dilute it to a greater and greater degree until it overflows the spillway. The same great dilution will take place when a full reservoir is drawn down to prepare for another flood.

The speaker has not the exact figures in mind, but the increased stage at low water on the Allegheny River and the consequent dilution, made possible by the proper operation of the seventeen reservoirs, would result in a reduction of both the hardness and the acidity of that river of something like one-half or one-third at extreme stages, with a marked reduction at other times.

The idea of the beneficial use of such reservoirs for many purposes other than the prevention of floods has resulted, as Mr. Grant has remarked, in the organization of a group of interested people in Pennsylvania; some of these are capitalists seeking investment and some are public-spirited citizens and representatives of civic bodies, who have come together on the common ground of desiring a sane development of the water resources of the State. They believe that the doing of one thing does not necessarily antagonize the doing of another, and that many great benefits can be obtained through private development with State regulation and co-operation. They are, therefore, interesting themselves in developing the water laws of the State.

KENNETH C. GRANT, ASSOC. M. AM. SOC. C. E. (by letter).—The seventeen reservoirs described in the paper have a combined capacity of about 60 000 000 000 cu. ft. The greatest flood that has ever visited Pittsburgh had a volume of only 26 000 000 000 cu. ft. above the flood stage. The recent flood of January 9th, 1913,* the fourth greatest in 48 years, had a volume of only 13 000 000 000 cu. ft. above the flood stage. Mr. Grant.

* Described in *Engineering News*, January 30th, 1913.

Mr. Grant.

It is not unreasonable to expect, therefore, even without the graphical proof described, that the reservoir system proposed by the Flood Commission will control floods at Pittsburgh effectively; and the discussions have not expressed any doubts of their effectiveness in this respect. It has been stated, however, that the reservoirs could not be used for flood control and for improving the low-water flow of the rivers as well. It is claimed that such a scheme is utterly impracticable, and has never reached a practical solution.

Reservoirs combining the purpose of flood control with those of water supply, water-power, and navigation are not an experiment. They have been built and are being operated successfully in other countries. The writer has had the opportunity of examining these works on two occasions during the last few years. Some of them he has described briefly* in his discussion of the paper by John H. Lewis, Assoc. M. Am. Soc. C. E.; others are to be found in Germany and Bohemia. A large reservoir constructed on a tributary of the Oder River, in Southeastern Germany, for flood control and water-power development, has given such good results during the 8 years since its completion, that another and larger project has been constructed, and is just about to be put in operation. Another large project is being constructed by the German Government on a tributary of the Weser, combining the purposes of flood control, improvement of navigation, and water-power development.

One of the discussions calls attention to the conditions that would obtain where the reservoirs were constructed on streams carrying a large quantity of acid from the mines. It is claimed that the greater specific gravity of the acid would lead to its concentration at the bottom of the reservoirs, and that, in the event of a great flood filling the reservoir, this acid would be swept out and become a menace to water supplies below. As a matter of fact, this very property of the acid, which would keep it at the bottom of the reservoir even when overflow took place over the spillway, simplifies the problem of avoiding the conditions of concentrated acidity that are described. By placing gates at the extreme bottom level of the reservoir, and others higher up, this acid could be released gradually from the bottom gates during flood times, to mingle with, and be diluted by, the large flood discharge. If found necessary, a simple arrangement of baffles would make this mixture complete.

The reservoir particularly referred to is near the mouth of Loyalhanna Creek. It is suggested that the dam be built farther up the creek, above Latrobe, in order to avoid storing the mine water, most of which originates on the lower portion of the drainage basin. There are several reasons why this should not be done, chief among which

* *Proceedings*, Am. Soc. C. E., for December, 1912, p. 1753.

Mr. Grant.

are the great reduction in the drainage area controlled, and the fact that the very conditions pointed out as so objectionable—the flushing out of the acid in the existing pools—would be done away with by locating the dam near the mouth and taking care of the acid as before suggested.

It must be borne in mind, moreover, that only two of the thirteen reservoirs proposed on the Allegheny Basin, those on Loyalhanna and Black Lick Creeks, are to be on streams that carry any appreciable quantity of mine drainage; and that the discharge from the other eleven reservoirs will dilute, to a large extent, any acid water that may come from these streams. Such dilution, of course, would not occur if the rainfall were local around the Loyalhanna or Black Creek basins; but that is just where the beneficial effect of the reservoir near the mouth of each creek would make itself felt, by holding back the acid at the bottom until a favorable time for its discharge.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1250

THE ECONOMIC ASPECT OF SEEPAGE AND OTHER LOSSES IN IRRIGATION SYSTEMS.*

By E. G. Hopson, M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Le Conte, W. C. Hammatt, Luther Wagoner, and E. G. Hopson.

In a report to the Comptroller of New York City, made by John R. Freeman, M. Am. Soc. C. E., in 1900, on the New York water supply, attention was drawn in a very clear and forceful manner to the enormous proportion of waste incident to the operation of a great city water-works system. The subject had been dealt with before by other engineers, and has been handled in a very comprehensive way by others since, but the writer did not recall at the time ever having seen the subject dealt with so comprehensively as in Mr. Freeman's report.

On page 38 of that report there is an interesting diagram showing the consumption of Croton water hour by hour during a typical week. By an ingenious interpretation of related, but more or less disjointed, bits of evidence, it was shown that of a daily delivery of 115 gal. to each inhabitant of the city, only about 40 gal. were really used and about 75 were wasted, that is, the proportion of use to waste was about 1:2.

It was further deduced that of the 75 gal. wasted, 65 was in all probability needless waste, and could be stopped by the adoption of proper measures. Naturally, the question arose as to whether it was worth while for a city to continue to lavish vast sums in the construction of new works, the greater part of the product of which

* This paper was presented at the Annual Convention at Seattle, Wash., and also at the meeting of December 4th, 1912.

would flow into the sea without benefit to any one, or whether it would not be better policy to devote some of this money to internal improvements in works already built, whereby savings equivalent in their effect to extensions of the supply system could be effected. Since the Freeman report was made much additional information has been gained on waste and its prevention in city water-works systems, and it has been shown that the amount of what was termed by Mr. Freeman needless waste is not quite so great as had been supposed. The question as to whether enforced economy in use is better policy than increasing the capacity of the system is still, to a large extent, a debatable one.

The reasons in favor of moderate consumption and avoidance of waste apply with even greater force to an irrigation system than to a city water-works system, in spite of the fact that the cost of the latter is relatively much higher than that of irrigation works. In a great city the cost of water-works is a comparatively light burden to the community, the expense to the individual of an unrestricted supply of pure water being one of the smallest items in his annual expense account. On the other hand, anything in the nature of restriction in use directly affects the personal convenience of each inhabitant, and is resented; he often prefers paying an extra trifle in order to enjoy not only a sufficiency but an excess.

With an irrigation system conditions are different. Usually, the supply is limited in quantity, and a waste in one direction is immediately reflected by straitened conditions in another. A system of irrigation work is designed to supply a definite quantity of water to each acre of land. The engineer makes certain allowances for waste and losses by seepage and evaporation. If his calculations are correct, the land receives a supply considered by him as sufficient, but not excessive. If, however, through some unexpected cause, the waste or losses are greater than were anticipated, less land can be brought under cultivation than had been contemplated, or farmers are compelled to get along with less water than had been considered necessary; hence the results are felt immediately and directly.

In the case of irrigation, as with a water-works system, losses can be classed as curable and incurable, and it is the writer's purpose to consider briefly those classes, as illustrated by works constructed by the Government in the Northwest during the last five or six years.

Roughly speaking, incurable losses in irrigation systems result directly through water lost by absorption in the beds of reservoirs and evaporated into the air. Curable loss lies almost wholly in that absorbed in the beds of canals and other conduits.

In the great storage reservoirs required for irrigation works it is obviously an economic impossibility to accomplish anything in the way of preventing absorption or seepage losses in their beds. Whatever losses result through this cause must be accepted as unavoidable. The engineer, however, must be prepared to accept responsibility for results, as his advice or decision on the all-important question of selecting or approving a reservoir site is the only safeguard against what may be disastrous loss if his judgment is ill-advised. For this reason the writer is illustrating the fundamental differences in conditions and results in four typical irrigation reservoirs built in the Northwest by the United States Government.

The East Park Reservoir is strictly a storage reservoir, built on a branch of Stony Creek, one of the Coast Range feeders of the Sacramento River. The dam site is a good one, being a notch in a great conglomerate dike or ridge that runs through the country in a north and south direction, and the dam is a solid masonry structure of the gravity type on an arched plan. The bed of the reservoir is practically wholly in the typical California shale. The dam was completed in 1910, and water was first stored in the winter and spring of 1910 and 1911. Weekly measurements are taken of the influent and effluent, the storage, and the rates of evaporation.

The maximum capacity of the reservoir is 45 000 acre-ft., and the maximum area of water surface is 1 690 acres. Table 1 shows the results in the season of 1910-11, the season being from November 1st to November 1st, in this and all the following cases.

TABLE 1.—East Park Reservoir, 1910–11.

	Acre-feet.	Percentage of influent.
Influent	65 400	..
Effluent and losses:		
Evaporation	7 100	11
Use, waste and surplus	58 300	89
*Seepage	0	0

*No appreciable seepage loss.

FIG. 1.—COLD SPRINGS RESERVOIR, OREGON.

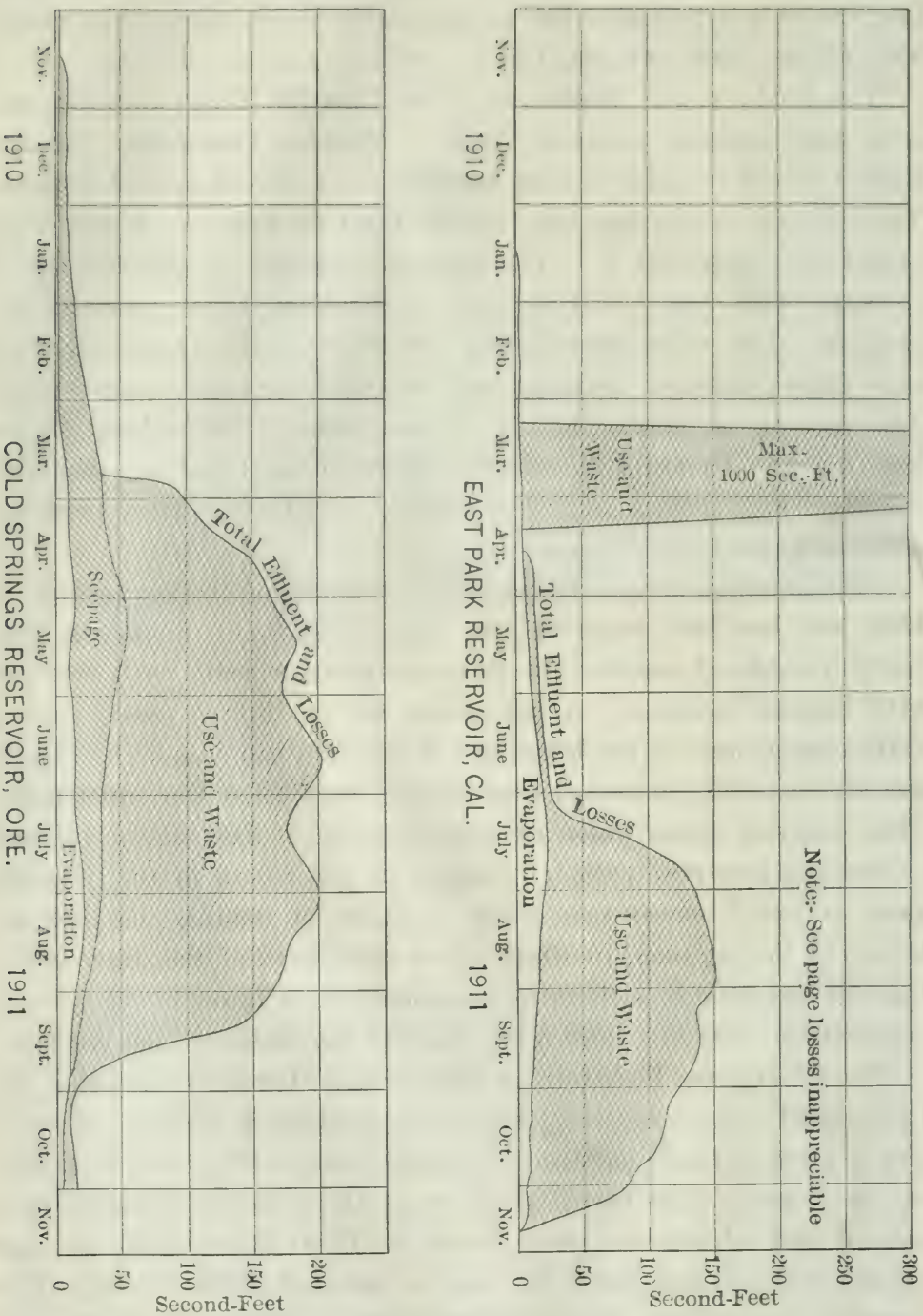
FIG. 2.—JOINTING 46-INCH CONCRETE PIPE, UMATILLA PROJECT.

This reservoir represents the highest condition of efficiency of any of the four described. The records fail to show any seepage loss, the only appreciable loss being that by evaporation; thus nearly 90% of the water entering this reservoir is available for use.

The Cold Springs Reservoir of the Umatilla Project, in Oregon, is a good average reservoir, from a Western standpoint. In the East it would probably not be regarded as a site of special promise. The dam is an earthen one, nearly 4 000 ft. long, of a maximum height of nearly 100 ft. The general structure of the country is volcanic, with vast overlying beds of stratified sands, gravels, and hardpan. The valley constituting the reservoir site is the outlet of some 200 sq. miles of drainage area with little or no ordinary run-off. The reservoir is supplied by a feed canal, some 25 miles long, diverting from the Umatilla River at times when the latter has available water. The capacity of the reservoir is 50 000 acre-ft., and its maximum area is 1 550 acres.

This reservoir was first placed in commission in the spring of 1908, and has been operated ever since. There are, therefore, four yearly records of results. In this case measurements were obtained with unusual accuracy, as practically all the inflow passed over a sharp-crested weir at the lower end of the feed canal, and the effluent was also carefully measured over another weir below the outlet gates. This reservoir shows losses ranging from 34 to 24% of the influent during the four-year period. Judging by the record of the past two years, it would appear that a fair condition of stability has been attained in the regimen, in which about one-fourth of the water entering this reservoir is subject to unavoidable loss through seepage and evaporation. Table 2 gives a summarized tabulation of the results.

The Clear Lake Reservoir, in California, situated just south of the California-Oregon line, is a feature of the Klamath Project. It occupies a great natural depression or sink, some 25 000 acres in extent, at the reservoir flow line. About one-half of the bed consists of a natural sink of alkaline water known as Clear Lake which for ages has received and evaporated the surplus waters of Willow Creek. This reservoir was built by the Government principally for the purpose of holding back the waters of Willow Creek, in order to facilitate the unwatering of lands marginal to Tule Lake, a body of water into which Willow Creek ultimately discharges. The reservoir was intended to


Nov.
Dec.
1910
Jan.
Feb.
Mar.
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May
June
July
Aug.
1911
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Nov.
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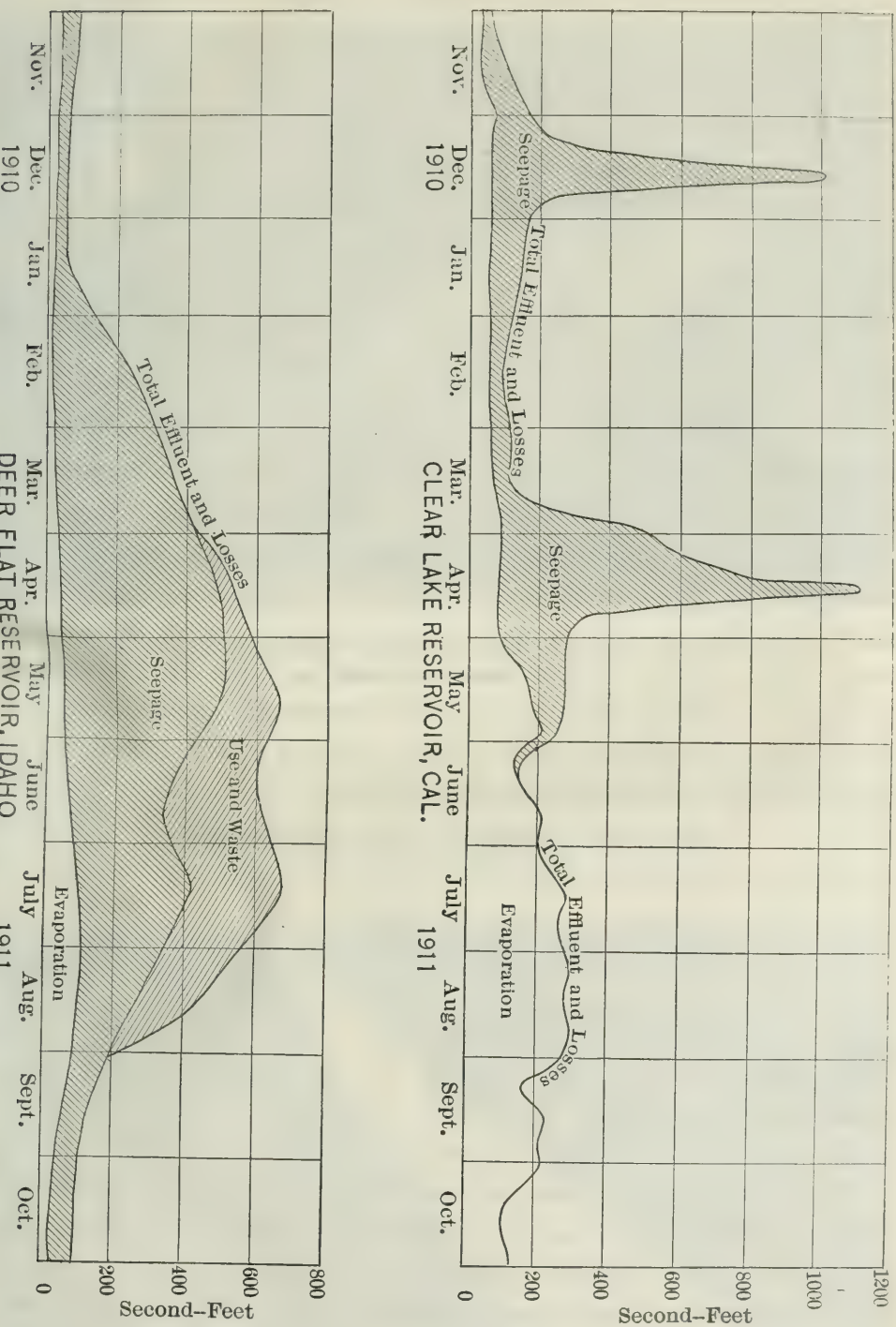
1000 Sec.-Ft.
Use and Waste
Total Effluent and Losses
Evaporation
Losses
Use and Waste
Note:- See page losses inappreciable
0
50
100
150
200
250
300
Second-Feet
EAST PARK RESERVOIR, CAL.
Total Effluent and Losses
Seepage
Use and Waste
Evaporation
COLD SPRINGS RESERVOIR, ORE.

FIG. 3.


Nov.
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July
1911
Aug.
Sept.
Oct.
Seepage
Total Effluent and Losses
Seepage
Total Effluent and Losses
Evaporation
0
200
400
600
800
1000
1200
Second-Feet
DEER FLAT RESERVOIR, IDAHO
Total Effluent and Losses
Seepage
Use and Waste
Evaporation
0
200
400
600
800
Second-Feet
CLEAR LAKE RESERVOIR, CAL.

Fig. 4.

TABLE 2.—Cold Springs Reservoir, Oregon.

Percentage of Losses Expressed in Terms of the Influent.

	1908.		1908-09.		1909-10		1910-11.	
	Acre-feet.	Percentage.	Acre-feet	Percentage.	Acre-feet.	Percentage.	Acre-feet.	Percentage.
Influent	20 366	..	42 820	..	61 526	..	72 273	..
Effluent and losses:								
Evaporation	2 400	12	4 295	10	5 333	9	6 252	9
Seepage	4 515	22	4 021	9	10 461	17	10 878	15
Use, waste, and surplus	13 451	66	34 504	81	45 732	74	55 163	76
Return flow	.....	..	865	..	503	..	182	..

combine the purposes of a great evaporating pan and a regulator of the diversion channel that diverts the discharge of Lost River from Tule Lake into Klamath River. More recent plans, however, have considered its possibilities as a source of irrigation supply. The capacity of the reservoir is enormous, as compared with the available water supply, being 450 000 acre-ft., with an area of 25 000 acres. The dam on Willow Creek is a rock fill structure, some 30 ft. high, which was completed in 1909. There are two years' records of the action of this reservoir, as given in Table 3.

TABLE 3.—Clear Lake Reservoir.

	1909-10.		1910-11.	
	Acre-feet.	Percentage.	Acre-feet.	Percentage.
Influent	141 000	....	225 000	.....
Effluent and losses:				
Evaporation	80 000	57	88 000	39
Seepage	48 000	34	24 000	11
Use, waste, and surplus	13 000	9	113 000	50

The rate of evaporation in this vicinity has been estimated at a little more than 4 ft. in an average year. It will be noted that evaporation is the principal loss in the Clear Lake Reservoir, as had been anticipated. The seepage losses during the first year were heavy, but, apparently, the marginal lands have filled up, so that the losses in 1911 were comparatively moderate. It is important to note that in a year of copious run-off, like 1910-11, as much as 50% of the

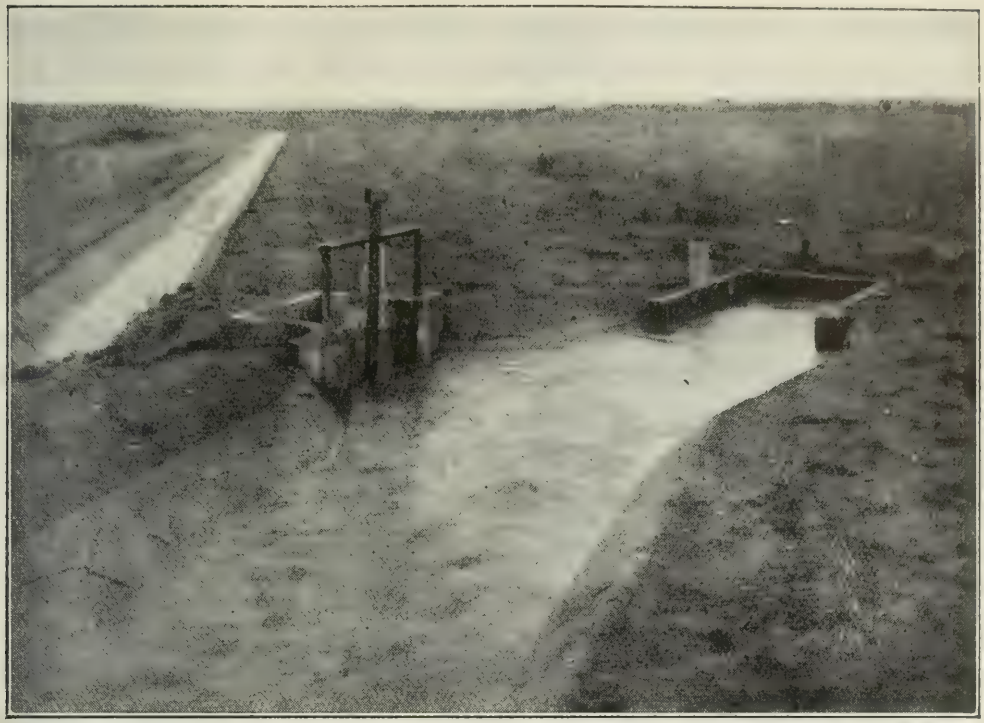


FIG. 5.—MORTAR-LINED LATERALS AND CONCRETE STRUCTURES, UMATILLA PROJECT.

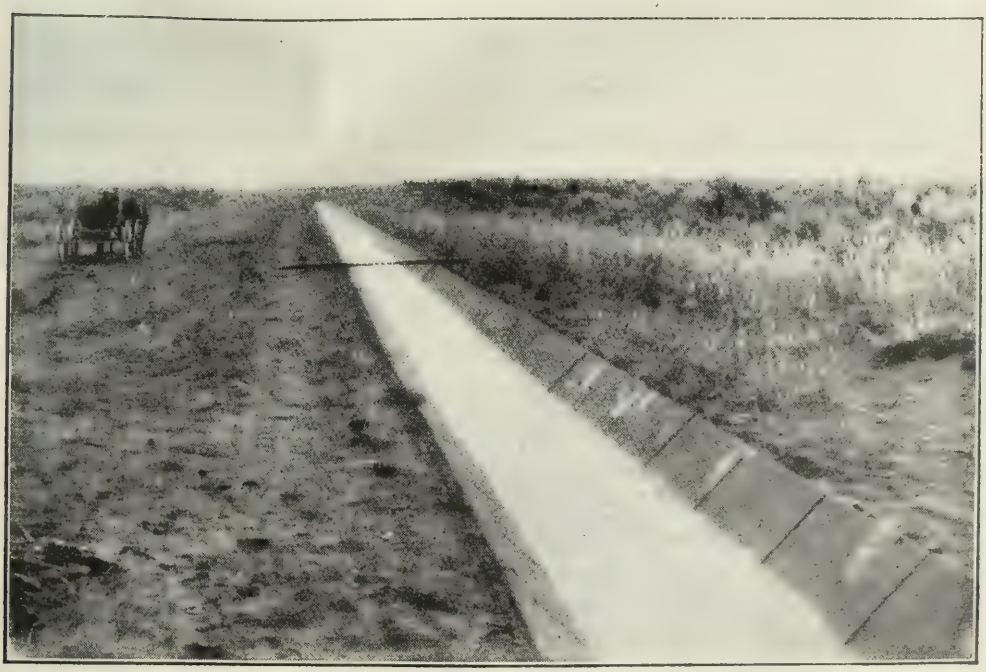


FIG. 6.—MORTAR-LINED LATERAL, UMATILLA PROJECT.

supply was subject to unavoidable loss or waste, which, in this case, was intentional, the principal purpose of the reservoir being the disposition of surplus water, rather than its conservation for use.

The Deer Flat Reservoir, a feature of the Boise Project, in Idaho, presents different natural conditions from the three preceding types. It does not occupy a natural drainage valley or sink, but, on the contrary, is situated on a flat saddle between the hills, the lower ends of which are closed by two earthen dams. It has a maximum area at high-water line of 9 250 acres, with a maximum capacity of 186 000 acre-ft. The reservoir derives its supply, as in the case of the Cold Springs Reservoir, through a feeder canal, known as the New York Canal, diverting from the Boise River some 10 miles southeast of Boise. The reservoir was first placed in commission in 1909, and has been in operation ever since. The bed consists in large part of silts, sands, and gravels, with a covering of from 3 to 5 ft. of soil. Seepage losses in this case have been pronounced from the outset, and constitute the bulk of all losses. When the reservoir was first placed in commission almost 90% of the water entering it was lost by absorption in the reservoir bed. In that year, however, the reservoir was only filled to one-tenth of its capacity. During the next two seasons larger and larger quantities of water were introduced, and the proportion of losses has fallen appreciably, but still remains exceedingly high. During the last season about two-thirds of the water entering this reservoir was subject to loss through evaporation and seepage. It may be expected that conditions will improve at this point as the adjacent and underlying strata of the reservoir gradually become filled by the constant application of water, but the extent and period of these ameliorating conditions are quite uncertain. A summarized tabulation of results is given in Table 4.

TABLE 4.—Deer Flat Reservoir.

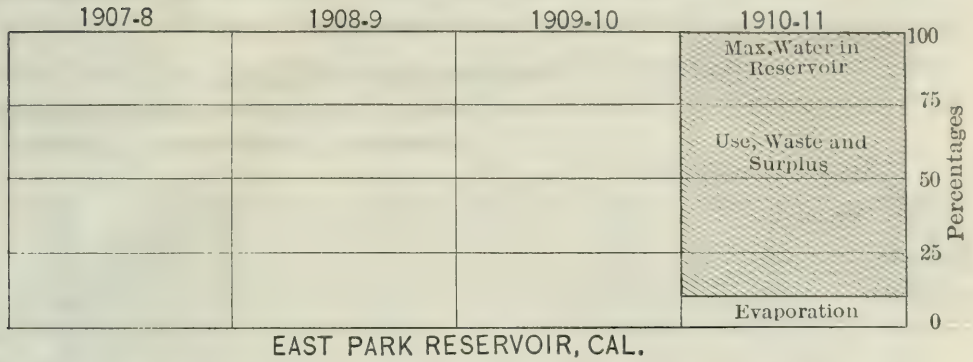
	1909.		1909-10.		1910-11.	
	Acre-feet.	Percentage.	Acre-feet.	Percentage.	Acre-feet.	Percentage.
Influent	64 000	.....	130 000	.....	230 000	.....
Effluent:						
Evaporation	4 000	6	18 000	14	20 000	9
Seepage	55 000	86	80 000	62	140 000	61
Use, waste, and surplus	5 000	8	32 000	24	70 000	30

The foregoing records, while incomplete and faulty in many respects, are among the best obtainable in a new country, and in any event are instructive. The general problem of reservoir losses is often given less attention by engineers than its importance warrants. In many cases the dam site is apt to monopolize attention, and an engineer accustomed to deal with reservoir sites in Eastern river valleys, where the adjacent water-tables are high and the losses are generally confined to evaporation, may be led to the commission of grave mistakes. A great deal has been said and written about return flow. One of the writer's earliest recollections in connection with reservoir studies was the discussions in the *Transactions* of the Society between Messrs. FitzGerald, Stearns, Fteley, and others, on ground-water storage of certain reservoirs in the East. Mr. FitzGerald's conclusions as to the general inadvisability of giving credit to the invisible storage of a reservoir are wise. Save under exceptional conditions, the writer doubts whether much, if any, additional draft can be made from Western reservoirs in excess of the visible storage. During the past four years the Cold Springs Reservoir has absorbed some 30 000 acre-ft. of water in its bed; it has apparently yielded back only about 1 500 acre-ft. The Deer Flat Reservoir has absorbed apparently 270 000 acre-ft., with little or no return.

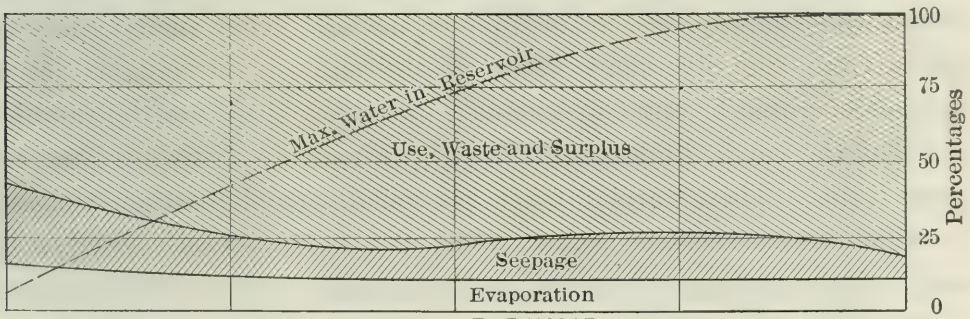
It is important to note that, in a reasonably good, representative, irrigation reservoir, such as Cold Springs, one-quarter of the water turned into it is lost; and that, apparently, under the most favorable circumstances, as at East Park, 10% will be lost.

The main lesson to be derived from these few illustrations is that the geologic structure of the site should be given the most careful consideration, as it is vital to determine in advance, as nearly as may be, the amount of reservoir losses, and whether they are likely to be of a permanent character.

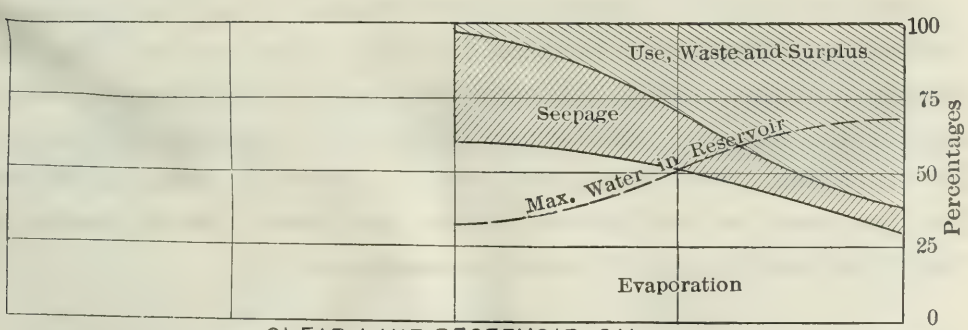
On the Umatilla Project, the cost of the irrigation works per acre of irrigable land is from $60 to $70; on the Truckee-Carson Project, about $40; on the Orland Project, about $50; on the Tieton, about $90; on the Sunnyside, about $50; and on the Klamath, from $30 to $40; say, an average of about $55. This is a fair indication of the general run of costs in large irrigation work in that part of the country, and is probably lower than the average costs on newer projects, either Government or private.



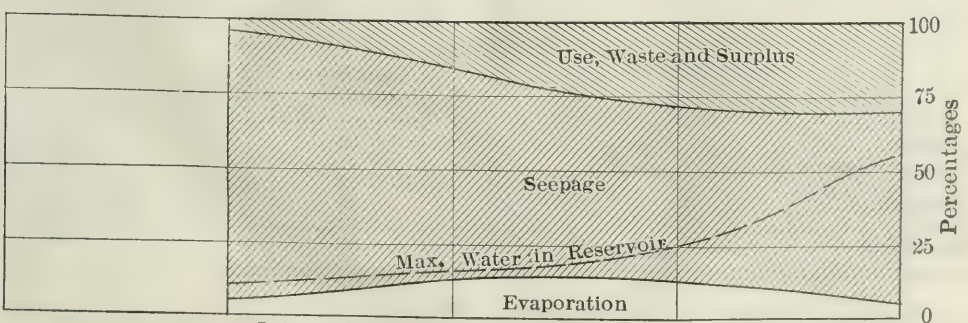
EAST PARK RESERVOIR, CAL.



COLD SPRINGS RESERVOIR, ORE.



CLEAR LAKE RESERVOIR, CAL.



DEER FLAT RESERVOIR, IDAHO

FIG. 7.

The various losses in the water-supply system, as expressed in percentages of water diverted, are as shown in Table 5.

TABLE 5.—PERCENTAGES OF LOSSES.

	Reservoir.	Canal losses.	Totals.
Umatilla	20	32	52
Truckee-Carson	0	41	41
Orland	8	23	31
Klamath	0	48	48
Tieton	0	24	24
Sunnyside	0	27	27

These losses, running from one-fourth to upwards of one-half of the whole supply, are, unfortunately, not all. They include only the losses from the diversions down to the end of the regular lateral systems operated by the Government; but below these are the ramifications of the small ditches built by the farmers to distribute water to their farms. These farm ditches are usually small earthen trenches, in which heavy seepage occurs before the water actually reaches the crop. In some cases farmers use water-tight flumes and pipes for their local distribution, but the proportion of these cases is as yet comparatively small, although on the increase. It has been estimated that seepage losses in the farmers' ditches on many projects is not less than 50% of the losses in the main canal and lateral systems. Allowing for the losses in the farmers' ditches not included in Table 5, the latter might be revised as shown in Table 6, it being understood that the losses in the farmers' ditches are merely the expression of individual opinion, not of actual measurement.

TABLE 6.—PERCENTAGES OF LOSSES.

	Reservoir.	Canal Losses.		Totals.
		Canals and laterals.	Farmers' ditches.	
Umatilla	20	32	15	67
Truckee-Carson	0	41	15	56
Orland	8	23	10	41
Klamath	0	48	15	63
Tieton	0	24	8	32
Sunnyside	0	27	7	34

Seepage losses on the Umatilla Project early assumed serious proportions owing to the sandy character of the soil and the gravelly substrata. With the unlined earthen ditches, as originally constructed, only about one-third of the water diverted reached its proper

destination. The works were costly, and the quantity of the supply was limited. Unless means could be found to lessen these losses, it was evident that the entire area could not be irrigated, and the building costs would not be wholly repaid.

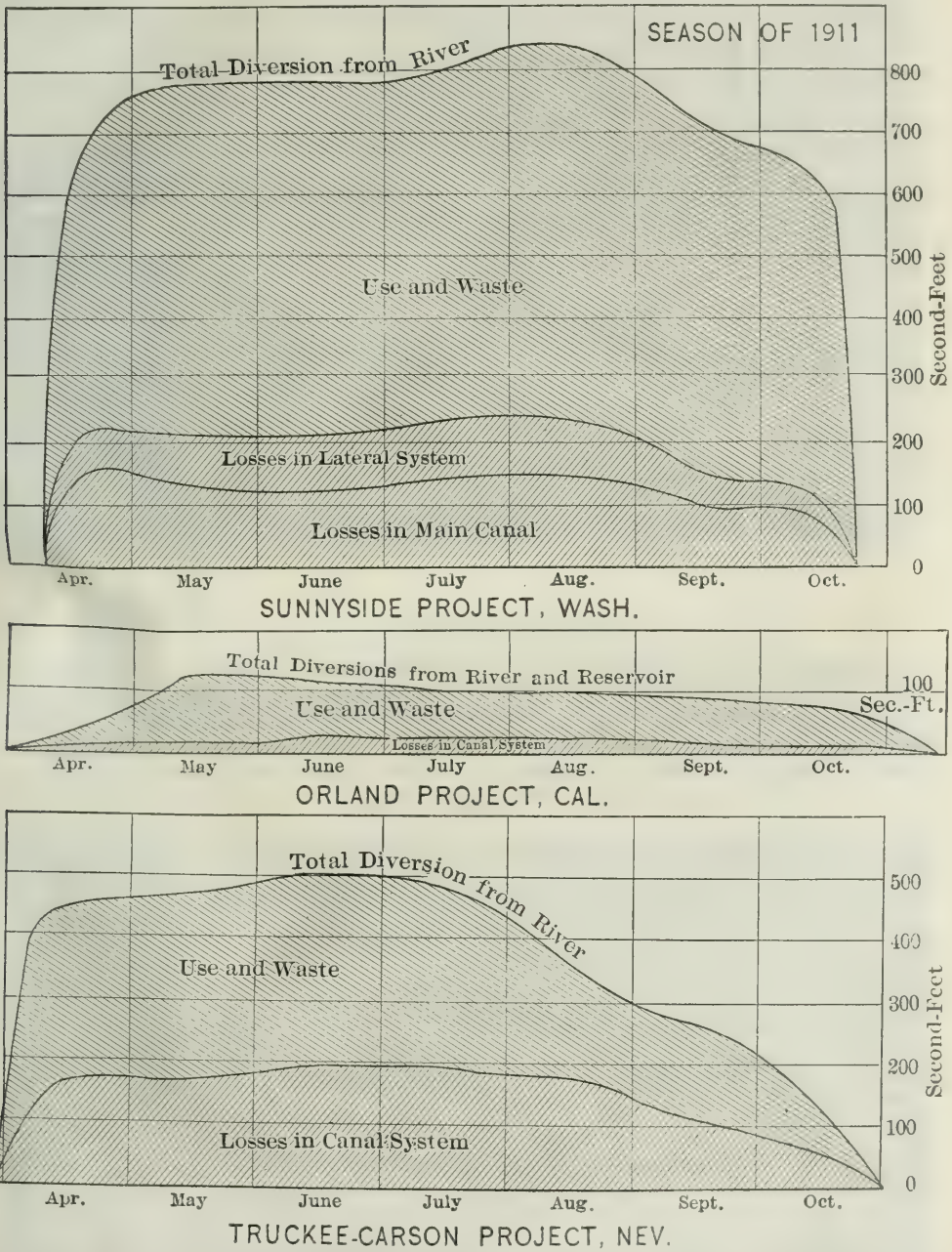
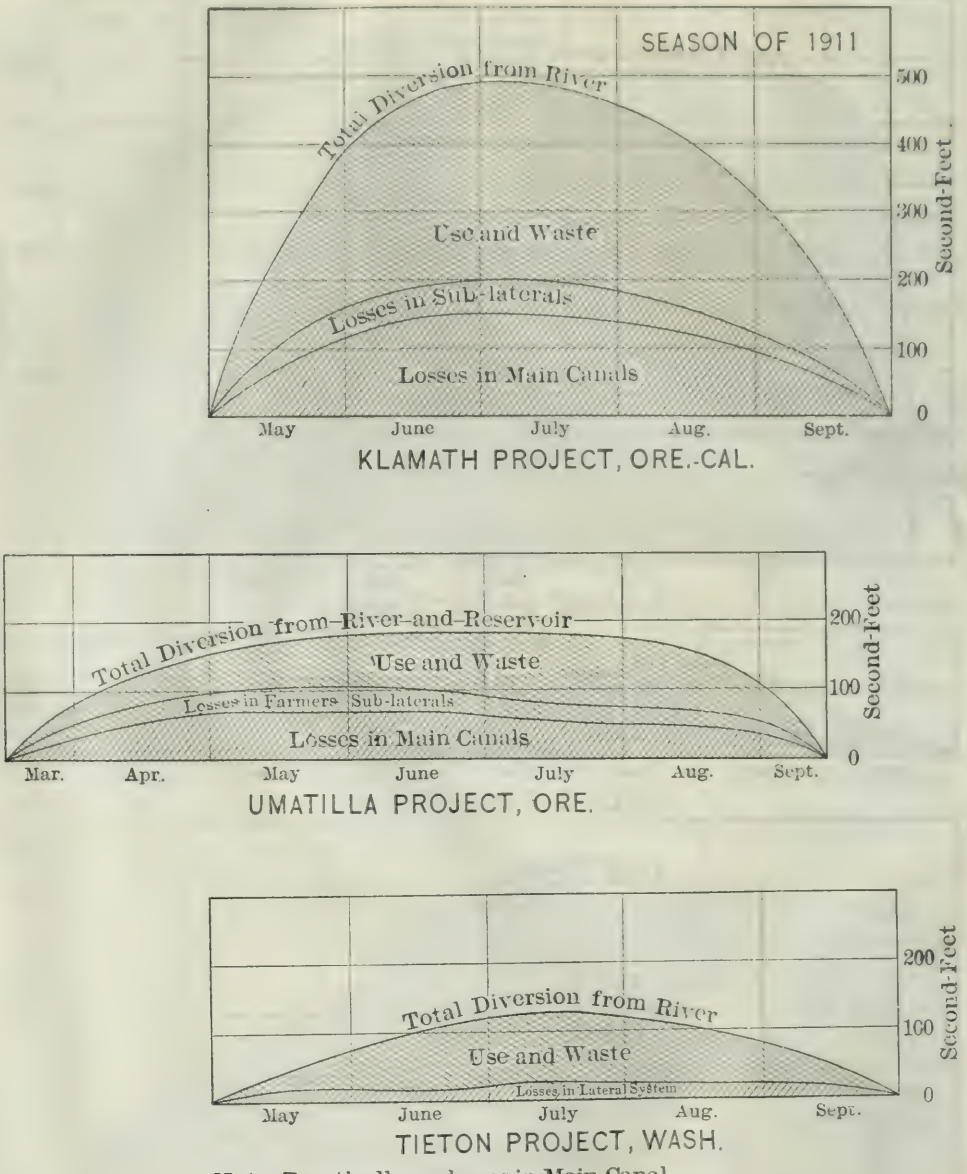


FIG. 8.

About equally severe proportional losses were found on the Truckee-Carson Project, in Nevada, and on the Klamath Project, in Oregon, but in both these projects there is more elasticity, due to their greater

available supplies, and, in the case of the Klamath Project, to the small aggregate quantity used.

Losses on the Tieton and Sunnyside Projects are probably much more satisfactory than in the average well-constructed project in that



Note:-Practically no losses in Main Canal

FIG. 9.

vicinity, due, in the first case, to the complete concrete lining of the main canal and the tight character of the substrata of the irrigated lands. In the Sunnyside Project, the relatively small canal losses are due mainly to the fine texture of the soils.

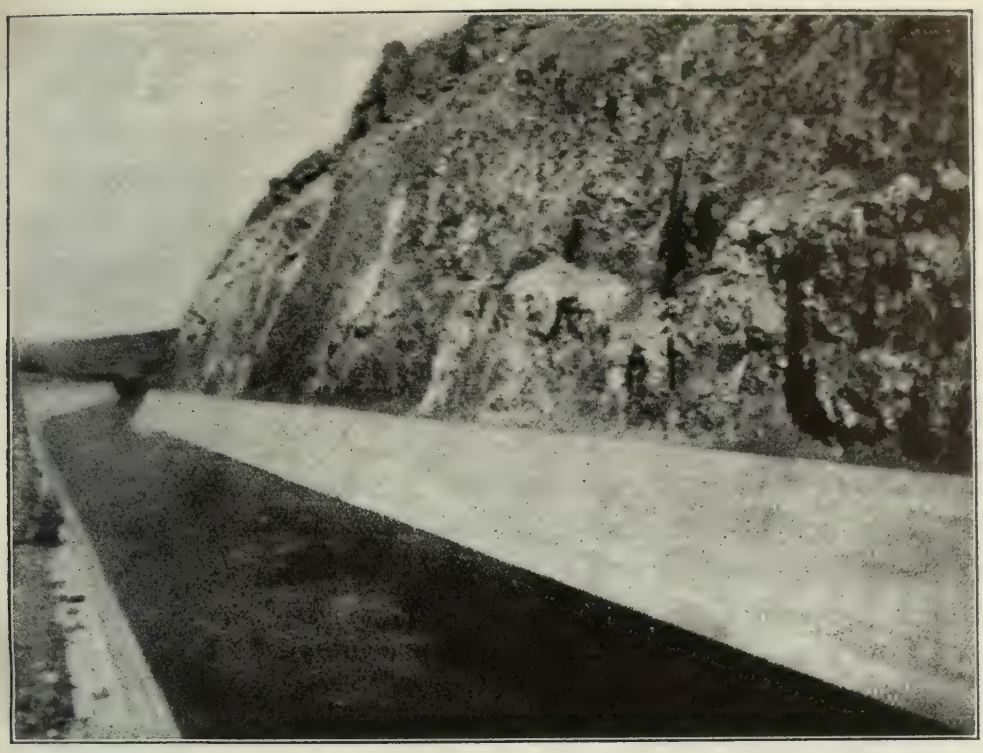


FIG. 10.—MAIN CANAL, TRUCKEE PROJECT, CONCRETE LINED.

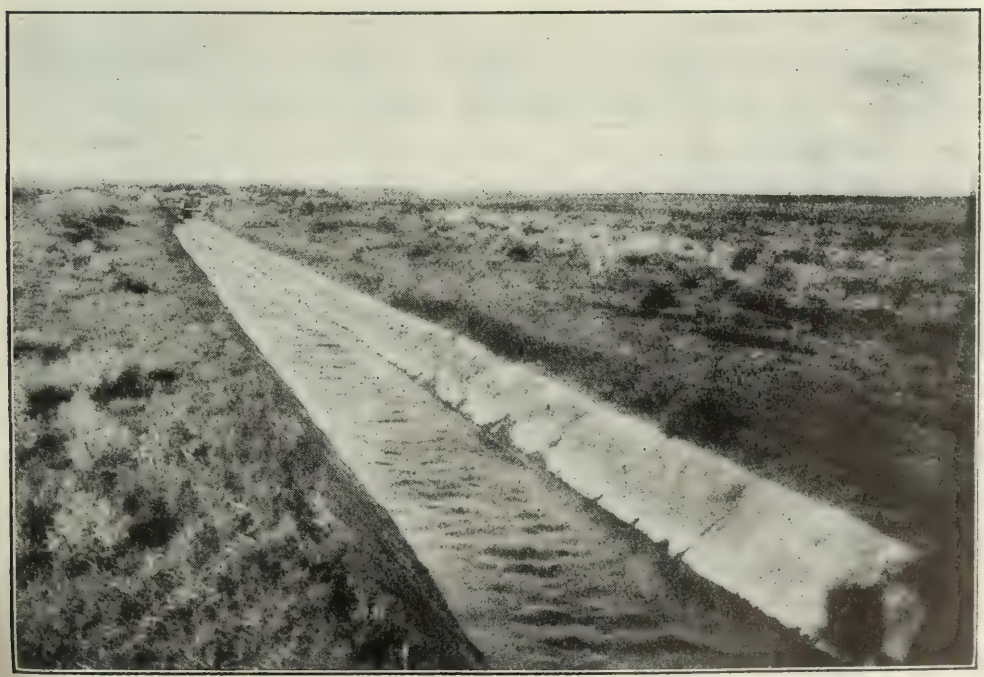


FIG. 11.—MORTAR-LINED LATERAL, UMATILLA PROJECT.

It is probably a fact that in the average project from 40 to 50% of the supply is lost by seepage in the beds of channels before it reaches the actual point of application. As the farmer is paying from $30 to $90 per acre for this water, the loss is very appreciable.

In Southern California valuable orchard lands have been under irrigation for a generation. Crop values have been very high, and, in many cases, the water supply has been so limited that effective measures toward conservation have been enforced. On many projects in that region the distributing channels are lined with concrete, or pipe is used liberally. The high values of lands and the scanty water supply have rendered these measures not only desirable but necessary. Strict economy in use has also been enforced, for the same reason, but, in the newer projects in the Northwest, where crops of lower values obtain, it has not hitherto been seriously regarded as feasible to resort to such expensive treatment. Conditions, however, have changed materially with regard to crop values, and many of the water supplies which appeared to be inexhaustible a few years ago are being rapidly fully appropriated, so that reasons for economy and waste prevention are becoming more and more cogent.

Some interesting experiments carried out under the auspices of the College of Agriculture of the University of California, in 1906, by B. A. Etcheverry, Assoc. M. Am. Soc. C. E., on various kinds of canal lining, including concrete, clay puddle, and oiled surfaces, are worthy of consideration. The object of these experiments was to determine relative costs and efficiencies of different classes of lining in reducing seepage and preventing the growth of vegetation. Without attempting to enter into the details of these experiments,* the general results showed that the concrete lining alone, although the most expensive, gave assured results. The oiling, as would be expected, is very much cheaper than any other treatment, costing only about one-quarter as much per square foot as concrete. During the first year it appears to be of some value in reducing seepage losses, measurements showing that the losses, as compared with those in an untreated earthen canal, are only about 40% of the latter. The oil seemed to be principally valuable in preventing a growth of vegetation. The clay puddle lining gave somewhat better results in preventing seepage

* "Lining of Ditches and Reservoirs to Prevent Seepage Losses," *Bulletin No. 188*, Agricultural Experiment Station, University of California.

than the oiled surface. The mortar and concrete linings, however, prevented from two-thirds to nine-tenths of the total losses, and, of course, entirely stopped the growth of vegetation.

Apparently, the effect of the oil treatment is only temporary, and a year or two afterward, a re-examination of the canals in which the experiments were made showed that the growth of vegetation in the oil-treated canals was equal to that in those untreated, and in all probability the seepage losses were also as great.

In 1910 and 1911, a lateral on the Umatilla Project was lined with mortar, 1 in. thick, and careful measurements were made to determine the losses. The lateral had been selected for lining on account of the very porous character of its bed and in order to reduce seepage loss. With the lateral closed at the ends by dams, measurements showed that the water surface lowered about 0.1 ft. each day in the lined ditch, and by applying this rate of loss to the canal system as a whole, making due allowance for velocity of flow, it was computed that the aggregate seepage loss in the project, if all the canals were lined, would be about 5% of the supply. With the unlined system the loss is close to 50 per cent. Subsequent measurements have confirmed the above, and, from these and other data, the conclusion has been reached, that seepage losses can be kept down to less than 10% of the amount diverted, if good linings are placed.

During the past two years much canal and ditch lining has been placed on Government projects in the Northwest. These linings are from 1 to 4 in. thick, depending on the size of the canal and the conditions. The heavy linings are of regular sand and gravel concrete having about 1 part of cement to 8 parts of sand and gravel. They are generally placed without forms, the sides of the channel being trued up and a rather dry mix being used. The cost has usually been about $6 per yd. The great bulk of the ditch lining, however, has not been of regular gravel concrete but of mortar, which is usually composed of 1 part of cement to 4 parts of sand. Before placing the mortar the ditches are carefully trued up by running a movable form or templet along their courses, and wetting and tamping the earth around the form. Immediately after the form is removed, the mortar is placed and kept damp until it has set well. It is jointed usually at about 4-ft. intervals in order to take care of temperature shrinkage. This

lining is done with much rapidity by experienced gangs. The materials are mixed in small portable gasoline-driven mixers, and the completed canals are kept full of water. The costs of work of this kind, carried out on a fairly large scale, for lining 1½ in. thick, reinforced at the top by an extra heavy curbing, run from 55 to 60 cents per sq. yd., inclusive of all administrative and engineering charges and of the earthwork. In general, the cost of the earthwork is about one-third of the entire cost.

Take, for example, a small lateral of the Umatilla Project lined in this way during 1911: The length was 12 400 ft.; the ditch dimensions were 4 ft. wide at the bottom, and 4 ft. deep, with side slopes of 1¼ to 1; the entire cost of the work averaged $1.05 per ft. Comparing a small ditch thus lined with an unlined one, the former will cost from three to four times more than the unlined ditch, but one of the economic advantages which the lined ditch possesses is the greater velocity of flow possible and the consequently smaller cross-sectional area of the channel. Another important advantage in lined ditches is the avoidance of drop structures. It is surprising what a large proportion of the total cost of ditch building goes into drop structures which are necessary in order to keep velocities below the eroding point in an earthen channel. With lined channels high velocities are not only possible, but desirable, in order to keep the channel clean.

As an illustration of what proportion the cost of structures in a distribution system bears to the entire expense of the latter, figures taken from the Orland Project in California are given. This territory is notably free from topographic irregularities, and the earth is firm and good for building purposes. The proportion of cost of structures, therefore, would be expected to be small.

The lateral system covers 14 000 acres, and includes 54 miles of ditches ranging in capacity from 12 to 75 sec-ft. Very little ditch lining has been placed, but the structures are all of concrete, the cheapest building material. The cost totals are as follows:

Excavation work	$64 376
Structures	57 632
Total	$122 008

The structures included in these cost totals comprise the following types:

Checks and drops	$20 885
Turn-outs	12 901
Bridges	9 972
Railroad crossings	6 924
Special structures	5 805
Spillways	1 143

Checks, drops, and turn-outs total $33 780, or 28% of the entire cost of the lateral system. All this cost could not be obviated by lining the system, but certainly a very large proportion could.

With a smaller cross-sectional area, the saving in drop structures, and the more direct and economical location possible in the lined ditches, the actual difference in cost per acre of land served by lined or unlined canals is comparatively small. It will generally be found to be less than $10 per acre, in many cases less than $5. If one takes into consideration the operating economics, the lined laterals have a distinct advantage by their freedom from breaks, seeped banks, and growth of vegetation in the channels, all of which should admit of a material reduction in operating costs. If these latter savings could be calculated from an investment standpoint and capitalized, any advantage in first cost of the unlined ditches would probably disappear, and a substantial margin be shown on the other side.

While considering canal lining, it would be well to give a little attention to the merits of pipe work in a distributing system. Large quantities of pipe have been used in the distributing systems of the Umatilla, Tieton, and Sunnyside Projects. The great bulk of this pipe is of concrete, both reinforced and plain, in sizes running from 54 in. down to 12 in. in diameter. The sizes below 24 in. have been usually made by the dry process, the reinforcement consisting of outside wire winding under tension. The larger diameters have usually been wet mixed, the pipe being manufactured in yards and hauled and laid like cast-iron pipe. Some of these lines of pipe are of great length, and work under heads running up to 110 ft. They have always given satisfaction, from every standpoint. A distribution system consisting wholly of concrete pipe would be undoubtedly the most satisfactory from an operating standpoint, and although the

FIG. 12.—TIETON MAIN CANAL, CONCRETE LINED.

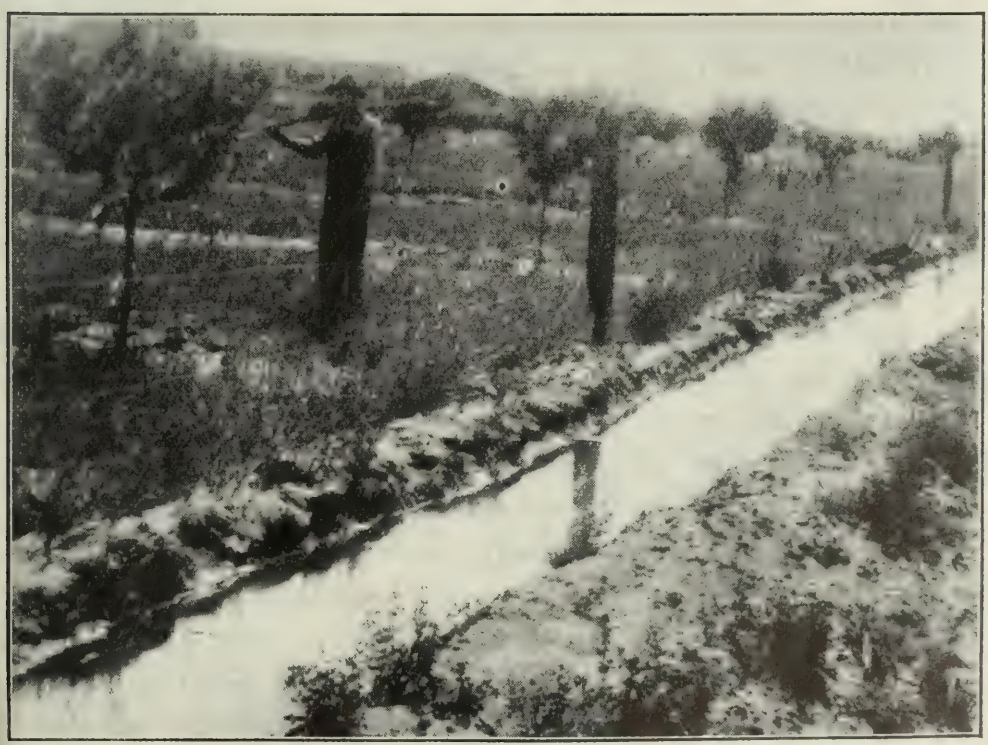


FIG. 13.—TYPICAL FARMERS' LATERAL, UMATILLA PROJECT.

first cost would be comparatively high, it might in the end prove to be more truly economical than the open-ditch system. With concrete pipe seepage losses are practically negligible. A number of tests of different lines of 4-ft. pipe, under operating conditions, show the following, all this pipe having a shell 3 in. thick:

	Length.	Head.	Average seepage per mile.
1	4 700 ft.	39 ft.	0.07 sec-ft.
2	5 400 "	28 "	0.05 " "
3	3 600 "	19 "	0.04 " "
4	9 800 "	85 "	0.20 " "

Apparently, the loss per mile in pipe of this size is nearly directly proportional to the head, and averages about 0.02 sec-ft. per mile for each 10-ft. head carried on the pipe. A pipe-distributing system of concrete throughout, under an average pressure head of 50 ft., with delivery to each 40-acre subdivision, would thus only lose about 1%, which is unimportant.

Taking the average of the six projects quoted, the average cost of the irrigation works would be $55 per acre, with an average combined loss in reservoirs and canals of about 50% of the entire water supply. Of the latter, about 6% is practically incurable reservoir loss; the remaining 44% has been classed as curable, that is, the great bulk of it can be cured or prevented if economic conditions render such action wise.

Should the ditch systems of these projects be wholly lined with concrete or pipe, the losses might be reduced from 44% to 10%, or less, a net saving of 34%, or, say, one-third of the whole supply. It is evident, therefore, that either the systems could be extended to cover about one-third more area, or if such land is not available, the works might be constructed of smaller dimensions and at less cost. In the case of works already built, the latter alternative is inapplicable, and is merely illustrative of what might have been done, but cannot be helped now. The lesson, however, should be applied to new work. In cases where new lands can be taken in under existing works, consideration should be given to the possibilities of extension by lining the present systems.

Suppose, for example, a project of 20 000 acres costing $55 per acre, or a total of $1 100 000; if, by lining the ditches, the irrigable

area can be increased to 27 000 acres, there will be first an additional cost for the new laterals with lining, which has been found to be about $18 per acre in a fairly difficult country, or, for the 7 000 acres, an additional construction cost of $126 000 will be necessary. Secondly, there will be the cost of lining the present ditch system, covering 20 000 acres, which, taken at $12 per acre, would mean an added charge of $224 000. The gross cost of the extended project, therefore, would be $1 450 000, or an average cost of $54 per acre. This, apparently, does not result in a material reduction in the acreage cost, but the great advantage lies in rendering available for profitable use the larger areas of land, the conservation of the water supply, and the avoidance of drainage evils referred to later. As a matter of fact, the process of extending a project already constructed with unlined earth canals, by the subsequent lining of ditches, is always much more expensive than if constructed *de novo* with the entire system lined.

In the case of a proposed large extension of the Umatilla Project, it is planned to line the entire canal system from the head-works down to the minor ramifications of the distribution system delivering to each 40-acre subdivision. At no point in the system will the water be exposed to avoidable seepage loss, and when the head-gates at the reservoir are opened, the Government will have the assurance that more than 90% of the supply will actually reach the cultivated fields.

Closely connected with the question of canal losses is the drainage problem. On nearly every irrigation project large and frequently increasing areas will be found subject to the rise of ground-water. The principal contributing influence in most cases is the seepage loss from the lateral systems, although a proportion, of course, is due to over-irrigation of the fields. On the Sunnyside Project, in Washington, some 4 000 or 5 000 acres of the best land were seriously affected a few years ago, large areas having been practically forced out of cultivation. In this case the Government was compelled to build a deep channel at a cost of some $340 000, mainly for the purpose of affording an outlet to the surplus water. On the Minidoka Project, in Idaho, the drainage feature is one of the most serious problems. At Umatilla the seepage water accumulating below the project in the Umatilla River has increased the summer flow some 100 sec-ft., and has rendered necessary the excavation of extensive drainage

ditches through the lower lands. At Klamath some $40 000 has been expended during the past three years on this account, and, on the Truckee-Carson Project, it is planned to expend not less than $400 000 in addition to the large sums already disbursed. There is no question that much relief from this increasing danger will be experienced by eliminating from the ground-water accumulations the bulk of the canal seepage. It is the writer's belief that, as time goes on, it may even be found necessary for legislatures to require canal systems to be lined or otherwise protected from seepage loss, not only in the interests of the investor and water user, but as a reasonable measure of conservation when water supplies are limited. As an engineering and business policy, it is well in the front rank, and should be considered by all who are building new works or operating and extending those already constructed.

DISCUSSION

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The curable and incurable losses of water in an irrigation system are certainly worthy of the most careful study and judicious consideration. The loss due to leakage from the reservoir bed is usually regarded as incurable, nevertheless, the site ought to be most carefully examined, geologically, before it is finally accepted.

Experience everywhere shows that when a large storage reservoir is built, the knowing ones always buy the water rights on the adjoining stream on the lower side. They know from experience that the leakage from the reservoir will swell the run-off of the adjoining stream. Where the stratification dips naturally from the reservoir site toward the stream in the adjoining water-shed, the leakage may be very serious. A notable case is that of the River Glyde, in Ireland, where the rainfall and run-off were being carefully observed by able engineers. Observations for 3 months in the rainy season showed a rainfall of 5.89 in. and a run-off of 9.35 in. This result gave rise to a great deal of merriment, at the time, but subsequent observations proved that the flood-waters from the higher adjoining water-shed found underground passages and escaped into the Glyde water-shed, thus swelling the observed run-off to abnormal dimensions. This was discovered by putting coloring matter in the upper stream and noting its appearance in the waters of the Glyde at a lower level, where the gaugings were being made. This simple experiment explained the whole mystery.

This only goes to show how fallacious it is to assume that the run-off records account for all the water that naturally comes from the direct water-shed. In point of fact, a large percentage of it may come from an indirect water-shed. A notable case is that of the Danube River; observations at the Iron Gate show that a large percentage of the summer flow goes through subterranean channels, and swells the summer flow of another stream in an adjoining water-shed.

It is quite common in every-day practice to pick out a dam site where a dike cuts across the valley, the presumption being that all danger of underground flowage is thereby avoided. This is true, but, at the same time, it makes a fine face for leakage to run along over and into the adjoining water-shed.

Mr. Hopson's estimate of a loss of 55% seems to be extravagant. In designing new works, it is generally customary to allow for a loss of one-third of the total, two-thirds of the water being delivered at its final destination. Of course, when the canals are first opened to service, the loss is very great (fully 55 to 60%); but it gradually grows less and less as the channels silt up. Shortly after the works

Mr. Le Conte.

are opened, the maintenance force begins the work of puddling the canal beds. This, of course, is usually done at times when the water is least needed for irrigation, and duties are not pressing. This puddling is kept up each year, until it is completed, when the total loss from seepage and evaporation is generally reduced from 60 to 33⅓%, but as low as 25% in some cases. In a majority of cases, the quantity of water saved by the canals being lined or unlined, but puddled, as usual, is not to exceed 25% at best, and may be much less; hence, lining is certainly questionable. Of course, in all cases where water is highly valuable, the scheme of lining the canals is entirely feasible and desirable; but, in pioneer countries which are being developed, the first cost is practically prohibitive, and, as a rule, that work is left to future generations.

The important point made by the author, however, that the cost of an unlined canal system and a lined canal system will be practically the same, certainly calls for full investigation. At present the writer is unable to see it in the light presented. The author also calls attention to the great saving in drainage troubles, brought about by a general and complete system of lined canals, laterals, and distributing ditches. There seems to be little doubt that the drainage troubles would be greatly mitigated; at present, they constitute the most distressing feature of irrigation works, the unsanitary conditions created and developed by the best types being notorious.

In most cases surface and sub-drainage take care of themselves. The natural drainage channels should be used for that purpose exclusively. They should be cleaned out and some little money should be spent in deepening, straightening, and correcting any natural defects.

The lamentable effect of excessive irrigation, on the one hand, and defective drainage, on the other, are beyond ordinary comprehension. When all the facts of any case are properly grasped, the drainage problem naturally develops into a veritable sink-hole for the waste of public money; hence the necessity for the exercise of the highest grade of good judgment.

In India the British Government was compelled to pass the most drastic laws to control irrigation.* On sanitary grounds, no water is allowed to be issued for autumn crops nearer than:

5	miles	from	a	military	post.					
1	mile	"	"	native	town	of	more	than	10 000	inhabitants.
½	"	"	"	"	"	"	"	"	5 000	"
¼	"	"	"	"	"	"	"	"	1 000	"

200 yd. from small villages.

*See discussion by Surgeon-General H. W. Bellew on paper "The Evils of Canal Irrigation in India, and Their Prevention," by T. H. Thornton, *Journal*, Society of Arts, Vol. XXXVI, p. 521 (Mar. 23d, 1888); and "The Injurious Effects of Canal Irrigation on the Health of the Population of the Punjab, and Their Remedy," by Surgeon-General H. W Bellew, *Journal*, Society of Arts, Vol. XXXVI, p. 640 (May 11th, 1888).

Mr. Hammatt.

W. C. HAMMATT, M. AM. SOC. C. E. (by letter).—The question brought out in this excellent paper is of great interest to irrigation engineers. It is really, however, a business proposition, as to how much expense is justifiable for the purpose of preventing, or diminishing, the losses and damage due to seepage from canals and ditches. Into this determination enter so many factors, that no rule or formula, however complex, can be made to cover the subject.

The quantity of seepage from canals has been the subject of many investigations, and is dependent on many conditions, which conditions will affect its cure. The writer knows of cases where, at certain seasons of the year, the seepage into a reservoir from its water-shed more than offsets the evaporation therefrom, so that the reservoir remains practically stationary under a 120° sun.

In both reservoirs and canals, the seepage loss is dependent on the depth of water in the canal, the breadth of the wetted perimeter, the soil through which the canal is excavated, the kind of subsoil and the distance thereto, the height of the ground-water, the slope of the country, the growth of weeds in the canal, the character of the vegetation outside of the canal, and many other factors. All water which seeps from the canal can be accounted for and traced to its destination. In some cases it has a flow through the soil, in a definite direction and at a determinable speed, toward some open watercourse or underground reservoir. The character of the soil through which the canal is cut may make a variation in the rate of seepage of from 0.3 to 1.6 vertical ft. per day—the limits of measurements made by the writer. Aquatic grasses and weeds require water for their growth, aid seepage by loosening the soil and facilitating the passage of water to the subsoil, and increase evaporation through their leaves. Measurements under the supervision of the writer have shown this to increase the natural seepage as much as 40 per cent. A certain quantity of seepage goes to supply the wants of near-by vegetation, thus creating a flow and a gradient in that direction.

The seriousness of the loss by seepage depends on various causes, among which are the following: The scarcity of water and consequent loss in crop value, due to lack of the quantity lost by seepage; the character of the soil, and the consequent tendency toward the rise of the ground-water; and the character of crops, and the effect on them of a high or low water-table.

In Southern California, where water is scarce and the crops consist mainly of citrus fruits, the value of the water for crop propagation will pay for an immense expense for seepage prevention. On the other hand, in Central California, where water is plentiful and the crops are grain, cereals, and alfalfa, and $5 per acre per year is a prohibitive price for water, very little expense for the prevention of losses is justifiable. It is seldom that the seepage from a canal is suf-

icient to water-log the soil. At the maximum rate the writer has een, namely, 1.6 vertical ft. in 24 hours, the seepage from a canal arrying 100 sec-ft. would be about sufficient per mile to raise the round-water 1 ft. in 900 acres. As the 100 sec-ft. would irrigate bout 16 000 acres, and as only about 10% of this would go toward rop propagation, the remainder staying in the soil, the small proportion f ground-water due to canal seepage is apparent. Consider also hat the farmer's tendency is to over-irrigate, especially where water s sold at a flat price per acre without regard to the quantity used, nd we have the reason for the drowning of so many good farming reas. Mr. Hammatt

B. A. Etcheverry, Assoc. M. Am. Soc. C. E., has made a great tudy of various linings for canals and ditches.* The data which he as compiled show a maximum cost of lining of about 6 cents per q. ft., and this is for thin mortar lining which is only adaptable to anals in solid ground capable of resisting the pressure, and with no ossibility of settlement. In most of the canal systems of Central California, the cost of the entire system, exclusive of the preparation f the land itself, has fallen to between $10 and $25 per acre of rrigated land. The cost of lining only the main canals and branches vould raise the cost of the systems from 25 to 50%, which would nly be justified by a continued scarcity of water.

The writer has only discussed the lining of canals for the preven- ion of seepage losses. Where the point of diversion is so much bove the point of use of the water that a steep grade and a swift low is obtainable, the lining of the canal—by reducing the co- fficient of roughness and by increasing the allowable velocity—will educe the section sufficiently to repay its cost, partly, if not wholly. Another element is the growth of weeds, which would be eliminated y a concrete lining.

To summarize: The question as to the lining of canal systems hould be decided by balancing, against the cost of such lining, the alue of the water lost, the damage done by that water, the cost of xcavation and of structures saved, and the elimination of the cost f canal cleaning.

Luther Wagoner, M. Am. Soc. C. E.—Under the speaker's direc- ion, a concrete lining was applied to a ditch near the Mokelumne River, in California, in 1912. To test seepage loss, a portion of the nlined ditch was filled 2 ft. deep with water; it settled at the ate of 1 in. per hour, and one day later there was a depth of water f 4 in. in the ditch. This loss, in comparison with the total quantity f water carried by the ditch, was small. The chief object of the ining was to avoid the numerous breaks caused by gophers. Mr. Wagoner.

* These data may be found in *Bulletin No. 188*, Agricultural Experiment Station, University of California, and in *Bulletin No. 44*, Department of Agriculture, Victoria, B. C.

Mr. Wagoner.

As there was no local supply of concrete material, the best bid obtainable was 9.7 cents per sq. ft. for a 1:10 gravel concrete, 2 in. thick. This was applied to the dressed banks and kept wet by watering with a hose. Later, a 1:2 grouting was washed over the surface. After two days, a section of the lined ditch was filled 2 ft. deep with water for use on the following section. One week later there was 1 ft. of water in the lined ditch, the loss of 1 ft. being seepage and use. It was not especially desired to cut down the seepage loss, but rather to stop perforation by gophers, and, for this reason, as well as to reduce costs, a 1:10 mixture was considered of ample strength to stand up on a 1 to 1 slope, and that it would be gopher-proof.

It was brought out at the Hetch Hetchy public hearing, before Secretary Fisher, at Washington, D. C., on November 25th-30th, 1912, that all unlined ditches sustain a considerable loss. F. H. Newell, M. Am. Soc. C. E., Director of the Reclamation Service, conceded that losses on the Turlock and Modesto, California, ditches and laterals might reach from 35 to 45% of the water turned into them, and that a 2-in. sand-cement lining might cut down the losses to 15 per cent.

The possible saving of from 20 to 30% of the water by lining would only be an economic possibility when water is high priced, or when it can be used for a high-priced crop like citrus fruits. For the ordinary crops, and present conditions in Central California, it does not appear as yet that there is any economic warrant for lining. As the wealth of the community increases, doubtless this will be done, largely for the purpose of extending the use of the water over a greater acreage, as the areas of good lands are greater than can at present be supplied with water.

Mr. Hopson.

E. G. Hopson, M. Am. Soc. C. E. (by letter).—In closing the discussion on this paper, the writer notes that some who have taken part therein have expressed the opinion that in certain localities, with which they are familiar, conditions would not warrant the cost of protecting ditch systems against seepage losses by lining. In the paper the writer has only attempted to indicate that, under certain conditions, such lining is warranted, and that the conditions requiring lining are tending to increase each year, judging the matter entirely from the economic standpoint.

In making this statement it is well understood that, with low-priced crops, reasonably tight soils and subsoils, and an abundant water supply, the extra expense of placing concrete in the canals is unwarranted. A few years ago such action would have been considered unwarranted in almost every case. The writer has endeavored to illustrate the changes in opinion that are being forced by more straitened conditions in water supply, as irrigation development proceeds and the available lands are being more fully utilized, giving as

an example an important project, the feasibility of which has mainly depended on the economic use of the water supply, only possible by the avoidance of waste by seepage in the beds of the canals. Mr. Hopson.

In placing concrete lining in irrigation laterals, attention is called to the necessity of making careful selection of the ingredients of the concrete. The writer has had experience in ditch lining which has not fulfilled the purpose of minimizing seepage, mainly due to the use of coarse aggregates in the concrete itself. Coarse aggregates give a concrete which, possibly, is entirely satisfactory, from the standpoint of strength and durability, but relatively porous. In selecting the aggregates, it is believed to be very important to obtain well-graded mixtures containing sufficient fine material to furnish a dense water-tight concrete, using the term "water-tight" in the relative sense. A little care in making the selection and grading can generally be expected to give good results.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1251

SPECIFICATIONS FOR METAL RAILROAD BRIDGES MOVABLE IN A VERTICAL PLANE.

By B. R. Leffler, M. Am. Soc. C. E.

With Discussion by Messrs. Augustus Smith, Charles H. Mercer, P. J. Reich, and B. R. Leffler.

The excellent paper and specifications for movable bridges by C. C. Schneider, Past-President, Am. Soc. C. E.,* though quite general, pertained mostly to the common swing bridge, or one which rotates about a vertical axis. The writer has felt the need of specifications covering railroad bridges movable in a vertical plane, which necessity was created by the third and fourth track work in progress on the railroad with which he is connected. The common swing bridge is not well adapted to more than two tracks. The writer knows of only two four-track swing bridges in operation.

There seems to be a real necessity among engineers for specifications covering movable bridges of this class. The engineer who has not given long and special study to this class, which is handled mostly by patentees, cannot give adequate consideration to the various designs presented to him under intense competitive conditions. These specifications are intended as an aid to his judgment.

It is not considered wise, at this time, to enter into a discussion of the relative merits of the various patented bridges, the purpose of the specifications being to aid in producing a first-class structure for any style which may be adopted.

Some unsettled technical questions are considered, such as stresses in wire ropes bent over a sheave, the rating and testing of electric

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 258.

notors, the character of the grooves for the lubrication of trunnions arrying heavy loads, the designing of keys and key-ways, etc.

The writer anticipates that the average mechanical engineer will ot agree with the views on stresses in wire ropes. He believes that nechanical engineers use methods which are too loose (under the guise f so-called practical experience) in designing machinery parts. As enerally used, wire rope is much over-stressed, principally by being ent over sheaves which are too small. Such practice may do where he rope is readily inspected and easily replaced, but wire rope for upporting counterweights in lift bridges should be designed by formu-as which take into account the leading factors affecting the life and trength of the rope. A large factor of safety should then be used.

The rating of electric motors is that adopted by the American nstitute of Electrical Engineers, in June, 1907. Some engineers specify half-hour rating, which usually means a motor of the crane type. Motors for mill work are now being made, and are superior to any other ype for strength and ruggedness; these are tested on the one-hour ating, and are suitable for bridgework.

Considerable care should be devoted to the design and workman-hip of grooves in large trunnions for lubrication. The grooves should e large and allow of being cleaned. Compression grease cups should e used.

The design and workmanship of keys and key-ways do not usually eceive enough attention, as keys come loose and cause damage and elay in the operation of bridges. Erectors sometimes use offset keys, nade in the field, for adjusting the relative position of machine parts; ut such keys are very objectionable. A key of minimum size, based n the diameter of the shaft and low unit stresses, has been specified.

Cut gear teeth are specified for wheels transmitting considerable ower. This is somewhat unusual; but, as most railroad bridges re not hand-operated, the resulting smoothness in the running of the nachinery is desirable. Cut gears add a very small percentage to ne total cost of a structure. The cutting of cast gears sometimes eveals defects which otherwise would remain hidden.

Two formulas are presented for the strength of shafting, axles, tc. This subject is not treated very clearly in works on machine de-gn. The use of the term "equivalent twisting moment" is confus-

ing. The formulas conform to the practice in structural designing of giving a value for tension or compression, and one for shear, respectively.

The writer believes that bridge engineers often specify too high a wind pressure. As usually specified for stationary structures, this includes an allowance for unknown lateral forces which are caused by a train moving over a bridge. Obviously, a smaller wind pressure should be specified for a bridge in motion. A pressure of 10 lb. per sq. ft. means, according to the formula, $P = 0.0032\ V^2$, a velocity of 56 miles per hour. The machinery should be able to hold the structure for a pressure of 15 lb. The wind pressure on a long bascule bridge is a large item.

Designers are sometimes too careless in their methods of designing machinery, relying mostly on rules-of-thumb or so-called experience. All resistance should be separately (and finally in their totality) considered. Coefficients should be adopted for the various sliding and rolling surfaces. The resistance of the moving span and attached parts should be reduced to a single force acting at the rack or in the operating cable. The motor torque for overcoming this resistance, and the machinery resistance, should be shown for all positions of the moving structure. The best method is to plot curves showing the torques, etc.; the time of opening, in 5- or 10-sec. intervals, should be plotted as abscissas, and the motor torque, resistance at rack, etc., as ordinates.

A moving structure is subject to some impact stresses due to its own motion, the magnitude of which cannot be found. The coefficients given simply express the writer's opinion.

No claim of originality is made for all parts of the specifications. The writer is largely indebted to Mr. Schneider's paper; to J. A. L. Waddell and J. L. Harrington, Members, Am. Soc. C. E., for workmanship and material for wire rope and attachments; and to others. To some extent his labors have been those of a compiler.

There is scarcely any first-class technical literature in the United States on the bending of wire rope.*

The writer has endeavored to make the specifications complete, but, of course, this was impossible. Some points are not covered; for instance, a specification should be framed to cover the design of segmental

*Attention is called to the article by Chapman, in the *Engineering Review*, London, October, 1908.

and track girders in rolling bridges, with special reference to taking care of the heavy concentrated load.

A rough test of the power required to open a double-track trunnion bridge of 159-ft. span, weighing, for parts in motion:

Machinery	156 877 lb.	
(Steel) Counterweight	538 313 "	Counterweight truss-link plate.
Span	797 009 "	Operating struts.
Concrete	2 570 000 "	
	4 062 199 lb. = 2 031 tons.	

TEST OF POWER:

Controller notches.	Volts.	Amperes.	
1	212	150	Time required to open = 2 min. 20 sec.
2	210	200	
3	210	200	
4	210	250	
5	210	200	
6	210	190	
7	201	350	
8	201	225	
9	201	375	
10	201	300	
1	219	60	Time required to close = 2 min.
2	219	60	
3	219	60	
4	219	75	
5	219	75	
6	219	75	
7	205	250	
8	205	225	
9	205	200	
10	205	200	
Average of readings of another test.	205	240	Time required to open = 2 min. 15 sec.
	216	93	Time required to close = 2 min.

SPECIFICATIONS FOR BRIDGES MOVABLE IN A VERTICAL PLANE.

Scope.

1.—These specifications are intended to cover bascule bridges, which are such as rotate about a horizontal axis; and vertical lifts, which are those in which successive positions are parallel.

2.—The specifications of The New York Central Lines for Steel Railroad Bridges, for 1910, shall apply to movable bridges, except as noted herein.

Manner of Bidding.

Parts Classified as Machinery.

3.—Drums, cylinders, eccentrics, trunnions and their cast supports, shafting, pistons, gear wheels, racks, boxings, bearings, couplings, disks, cast sheaves and wheels, worm gearing, valves, pins about the axis of which the connecting members rotate, whistles, ram screws, end bridge-locks, rail-locks, indicators, cranks, axles, hooks, wrenches, and similar parts of machinery which require machine-shop work, shall be classified as machinery and be paid for at a common price per pound. Electric motors are not classified as machinery.

Sheaves.

4.—The large sheaves of vertical lift bridges, the webs and diaphragms of which are built up with plates, angles, and rivets, shall be paid for at a separate price per pound of finished weight, including casings and fastenings to trunnions.

Air Compressor Boilers.

5.—Air compressor tanks and steam boilers shall be paid for at a separate price.

Wire Ropes and Cables.

6.—Wire ropes and cables shall be paid for at a separate price per pound.

Sockets, Pins, Levers, etc.

7.—The sockets, pins, equalizing levers, and cable attachments to the trusses and counterweights shall be paid for at a separate price per pound.

Structural Steel Parts.

8.—Structural steel supporting the machinery proper, counterweight frames, counterweight trusses, towers, and links, shall be classified as structural steel and be paid for at the same price per pound as for the span itself.

9.—Structural steel which can be fabricated by the common shop methods as punching, reaming, drilling, shearing, planing, etc., as is usually done for stationary structures, shall be classified as structural steel and be paid for at the same price per pound as for the span itself.

Segmental Girders.

10.—Segmental girders in rolling bascule bridges and the horizontal girders on which they roll shall be paid for at a separate price per pound. This does not include any bracing, floor system, or other structural members which may be attached.

Electrical Equipment.

11.—Electrical equipment, such as wiring, switch-boards, controllers, lights, blow-outs, cut-offs, solenoids, switches, motors, etc., shall be paid for on a lump-sum basis.

12.—Cast-iron parts used in counterweights shall be paid for at a separate price per pound.

13.—Concrete in counterweights shall be paid for at a price per cubic yard in place.

Extra Parts, etc.

14.—It is to be understood that if any extra parts are needed, or any question arises, all difficulties shall be settled on the pound price basis as quoted and accepted for the parts in question.

General Details of Designing.

Self-Centering Devices.

15.—Self-centering and seating devices shall be used on the free ends of the moving span. Holding and forcing-down devices shall be used for the free ends of each truss.

Rail-Locks.

16.—Designs for bridging the gap between the shore rails and moving rails shall be furnished by the Railroad Company. Loose rails will not be allowed.

Air Buffers.

17.—Air buffers shall be furnished at the free ends of the moving span.

Counterweights.

18.—The counterweights shall be easily adjustable. Usually, this shall be done by adding or taking away cast-iron parts, or small concrete blocks.

Stairways.

19.—Metal stairways, with $1\frac{1}{2}$-in. hand-rail, shall be provided, for access to the machinery, trunnions, and counterweights.

Girders in Rolling Bridges.

20.—The reinforcements of webs in the segmental girders and track girders of rolling bridges shall be symmetrical about the center planes of the webs. The center planes of the segmental webs shall coincide with the corresponding center planes of the webs of the track girders.

Coefficients of Friction for Moving Span and Attached Parts.

21.—In calculating the resistances to be overcome by the machinery, the resisting forces shall be reduced to a single force acting between the pinion and operating rack, or in the operating cable. In determining this force, the following coefficients shall be used in starting the span, and, except for the stiffness in cables, shall be reduced one-half after motion is begun:

For friction on trunnions	$\frac{1}{8}$
For rolling friction of rolling bridges	$\frac{1}{12}$
For stiffness in cables	$\frac{1}{200}$

Losses in Operating Machinery.

22.—In figuring the machinery losses between the operating rack or operating cable and the motor, the following coefficients shall be used: for the efficiency of any pair of gears, 0.94; for journal friction, 0.07. The losses of any worm gear shall be taken at 30% for an angle of thread 20° or more.

Time to Open.

23.—The time to open the bridge after the ends are released shall be as specified on the proposal drawing.

Inertia.

24.—The force necessary to overcome the inertia and produce acceleration and retardation for the time of opening shall be considered. The machinery shall be capable of stopping the bridge in 6 sec.; for this purpose, the coefficient of friction in the friction brake shall be taken at not less than 25 per cent.

Impact in Structural Parts.

25.—In calculating the dead-load stresses in the moving structural parts, for the various positions of the open bridge, such stresses shall be increased 25% as allowance for impact. For stationary structural parts (as towers, and supporting girders in rolling bridges), to which moving parts are attached, or on which such parts roll, 15% of the static load shall be added as impact.

26.—In structural steel parts, where a percentage of the dead load or static load is added for impact, the unit stresses for stationary structures shall be used; the impact percentages are an allowance similar to that provided by an impact formula for stationary railroad bridges.

Impact for Machinery Parts, etc.

27.—The allowance for impact in trunnions, cables, cable attachments, and machinery parts is taken care of by lowered unit stresses.

Wind Pressure.

28.—The least wind pressure to be assumed in proportioning the machinery or moving parts shall be 15 lb. per sq. ft. on the exposed surfaces of the moving parts as projected on any vertical plane. The machinery shall be strong enough to hold the moving parts in any position for this pressure, and be capable of opening the bridge in the specified time at 10 lb. per sq. ft. wind pressure.

28a.—On the ordinary open-floor bridge with ties, the exposed surface to wind shall be taken as equal to 80% of a full quadrilateral the width of which is the distance from center to center of trusses and the length of which is that of the moving span.

Detailed Drawings.

29.—The Contractor shall make complete detailed drawings of the machinery, so that any other shop can take them and duplicate the machinery. No reference to patterns or individual shop practices will be considered in lieu of the complete drawings. These drawings shall show a general outline of the assembled machinery. The drawings shall be made on tracing cloth, each sheet 24 by 36 in. in outside dimensions. These drawings shall become the property of the Railroad Company on the completion of the job.

Outline Drawing of Machinery.

30.—The Contractor shall furnish an outline drawing of the machinery, on which are shown the forces acting on the gear teeth, the twisting moment and bending moment on shafts, and other necessary information for checking the strength of the machine parts. A tabulation of the formulas and methods of calculation shall be shown complete enough to allow them to be checked.

Torque Curves.

31.—The Contractor shall show by a drawing of curves the torque to be exerted by the motor or prime mover, as follows:

1. A torque curve for acceleration and retardation;
2. A torque curve for the frictional resistances;
3. A torque curve for any unbalanced condition of the structure;
4. A torque curve for the wind load;
5. A torque curve showing the greatest combination of resistances acting at any one time.

In figuring the friction at starting (this being twice the running friction), no acceleration of the moving mass shall be considered. This friction shall be considered as reduced to the running friction in the first second after the power is applied.

Center of Gravity.

32.—The Contractor shall check the location of the center of gravity of the moving span, including all parts attached thereto, and also the location of the center of gravity of the counterweight, including counterweight girders and trusses, by computations based on accurate weights calculated from shop plans. He shall submit duplicate sketches and copies of these computations accompanied by weight bills to the Railroad Company for approval.

Hand Operation.

33.—All bridges shall be equipped with hand-operating mechanism. The number of men and the time required to operate shall be estimated on the assumption that the force one man can exert on a lever is 40 lb. with a speed of 160 ft. per min. developing about $\frac{1}{5}$ h.p. For calculating the strength of the machinery, the power of one man shall be assumed as 125 lb., but 150 lb. shall be the minimum used and applied to the extreme end of a lever.

Operating Machinery.

34.—The parts shall be simple in design, and easily erected, inspected, adjusted, and taken apart. The fastenings shall hold the parts in place securely after they have been set.

Kind of Material.

35.—Rolled or forged steel shall be used for bolts, nuts, keys, cotters, pins, axles, screws, worms, piston rods, trunnions, and crane hooks, if any.

36.—Trunnions, pins, and shafting more than $3\frac{1}{2}$ in. in diameter shall be of forged structural steel. Shafting $3\frac{1}{2}$ in. or less in diameter may be of cold-rolled steel.

37.—Forged or cast steel shall be used for levers, cranks, and connecting rods.

38.—Cast steel, or forged steel, shall be used for couplings, end shoes, racks, toothed wheels, brake wheels, drums, sheaves, and hangers where the supported weight will cause tensile stresses. Large sheaves may be built of structural steel.

39.—Pinions shall be made of forged steel, and cut from the solid metal.

39a.—Pinions shall have not less than fifteen teeth.

40.—Sockets used for holding the ends of wire ropes shall be forged without welds, from the solid steel.

Cast Iron.

41.—Cast iron may be used in boxes for shafts 2 in. or less in diameter, and which obviously carry light loads. Other boxes shall be of cast steel.

42.—Cast iron may be used in eccentrics, cylinders, pistons, fly-wheels, and parts of motors which are usually made of cast iron. Cast iron shall not be used for any trunnion or axle support.

Metal for Bushings.

43.—Phosphor-bronze, brass, and Babbitt metal shall be used for the bushing or lining of journal bearings and other rotating or sliding surfaces, to prevent seizing.

44.—Phosphor-bronze, only, shall be used for bushing for the trunnions of bascule and lift bridges, or in any large bearing carrying heavy loads.

45.—The bushings for large bearings, such as for trunnions and similar parts, shall be held from rotating in their casings. The force tending to cause rotation shall be taken as one-eighth of the load on the trunnion or bearing, and as acting tangent to the surface between the back of the bushing and casing. It shall be practicable to take out the bushing when the trunnion is slightly lifted.

Castings.

46.—Castings which are to be attached to rough unfinished surfaces shall be provided with chipping strips. The outer unfinished edges of ribs, bases, etc., shall be rounded off, and inside corners filleted.

Bolts and Nuts.

47.—Bolts and nuts, up to $1\frac{1}{2}$ in. in diameter, shall have U. S. Standard V-threads. Nuts and exposed bolt heads shall be hexagonal in shape, and each nut shall be provided with a washer. If the nut will come on an inclined surface, a special seat, the top surface of which is at right angles to the bolt, shall be cast or built up to receive the nut. Bolt heads which are countersunk in castings shall be square.

48.—Nuts which are subject to vibration and frequent changes of load shall have locking arrangements to prevent the gradual unscrewing of the same. If double nuts are used for that purpose, each nut shall be of the standard thickness. Nuts shall be secured by split pins put through the bolt.

Screws

49.—Screws which transmit motion shall have square threads.

Tap-Bolts, Set-Screws, etc.

50.—Tap-bolts and stud-bolts shall not be used, except by special permission.

51.—Set-screws shall not be used for transmitting torsion to shafts or axles. They shall be used for holding keys, or other light parts, in place.

Collars.

52.—Collars shall be used wherever necessary to hold the shaft from moving horizontally. Each collar shall have at least two set-screws at an angle of 120 degrees.

Shaft Couplings

53.—Shaft couplings, unless of the flexible kind, shall be of the flange type, or split muff with bolt heads and nuts countersunk.

54.—Couplings shall be keyed to shaft.

Keys.

55.—Gib-head or hooked keys shall be used for keying machinery parts to shafts, except where otherwise shown. The keys shall have the proportions shown in Fig. 1, in which d is the diameter of the shaft

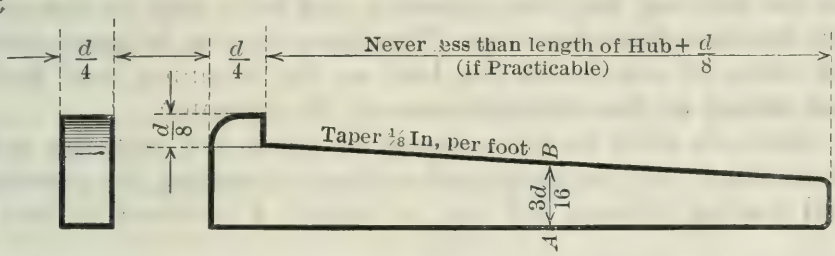


Fig. 1.

$A\,B$ is a mid-section of the tapered length. The sides shall be parallel.

56.—If the foregoing shape of key gives unit stresses in shear or bearing exceeding those in the table of allowable unit stresses, its section must be increased.

57.—The key shall be sunk in grooves in both hub and shaft. The finish of the grooves and key shall be such as to give a full bearing on all four sides, except as taper of key will not allow.

58.—If practicable, the groove in the shaft shall be made long enough to allow the key to be inserted without moving the wheel sidewise. After the key is firmly seated, the groove shall extend beyond the point of the key a distance not less than $\frac{3}{8}\frac{d}{}$ to allow for future tightening; the clear distance between hub and hook of key shall not be less than $\frac{d}{8}$.

59.—The depth of the groove in the shaft shall be $\frac{3}{40}\frac{d}{}$, measured at the side of the groove.

60.—In the case of large shafts carrying heavy parts, two or more keys of special design shall be used. In such cases, the matter shall be taken up with the Engineer, for special study.

61.—The foregoing requirements for keys and key-ways are for major machinery parts, the use of which is intended to develop the full torsional strength of the shaft. For minor parts, the keys and key-ways shall be proportioned for that size of shaft in which torsional strength would be developed by the minor parts.

Set Screws for Keys.

62.—Keys shall be held in place by set-screws.

Hub.

63.—If practicable, the length of the hub shall be not less than $2d$. Its thickness shall be not less than $\frac{d}{3}$. The hub shall have a light driving fit.

64.—The groove in the hub shall be made on the center line of an arm.

65.—Hubs shall be bored truly at the center of the wheel.

Keys in Trunnions.

66.—For trunnions and similar parts, which are designed chiefly for bending and bearing, the keys, key-ways, and bolts shall be designed to hold the trunnion from rotating. The force tending to cause rotation shall be taken at one-fourth the load on the trunnion, and shall be taken as acting at the circumference of the trunnion.

Journals.

67.—Journals shall be proportioned to resist, not only the various stresses to which they are subjected, without exceeding the permissible fiber and bearing stresses, but also to prevent a tendency to heat and seize.

Bushings.

68.—Steel bearings carrying steel shafts or journals shall be lined with bronze or brass. If shafts are 3 in. or less in diameter and of a slow motion, Babbitt metal may be used. Bearings of steel on steel for moving surfaces will not be allowed.

68a.—Divided journal and trunnion bearings shall be used, and the cap shall be fastened to the base with turned bolts recessed into the base. The nuts and heads shall bear on finished bosses cast on the bearing.

Boxes.

69.—In cast-iron boxes carrying light shafts, no lining is needed.

70.—The bearings of shafts shall be placed as near to the points of loading as possible.

71.—The foot-steps of vertical shafts shall be of axle or tool steel and shall run on bronze disks.

Lubrication.

72.—Provision shall be made for the effective lubrication of journals or any other sliding surfaces. Closed oil or compression grease cups shall be used. Grooves shall be cut in the surface of the trunnion to provide for the proper distribution of grease or oil.

Grease Grooves.

73.—The grooves in large trunnions shall approximate to a U shape the size shall be such that a wire $\frac{5}{16}$ in. in diameter may lie wholly within the groove. The edge of the U shall be rounded to a radius of $\frac{3}{16}$ in.

74.—The grooves shall be straight, running parallel to the axis of the trunnion. They shall be not less than three in number, and located so that all parts of the bearing surface of the bushing will be swept by the contained lubricant, in an opening, and in a closing of the bridge. The grooves must allow of being cleaned with a wire.

Grease Cups.

75.—In any trunnion bearing, or similar heavy bearings, strong compression grease cups shall be used for the grooves.

76.—Oil and grease ducts shall be located so that the lubricant will low by gravity toward the bearing surface.

77.—Dust covers shall be provided for principal bearings, in par- Dust Covers. icular for trunnions.

78.—Line shafts, extending from the center of the bridge to the Shaft Supports and Couplings. nd, shall not be continuous, but shall be connected with claw coup- ngs. Each length of shafting shall rest in not more than two bearings, ith the couplings close to the bearings.

79.—If shaft supports are connected to the floor-beams, in bridges aving long panels, intermediate supports shall be used; these shall e adjustable, and are intended merely to prevent the shaft from agging.

80.—Equalizing gears or devices shall be used to insure equal ac- Equalizing Gears. ion at the pinions and operating racks.

81.—The unsupported length of shafts shall not exceed $L = 80\sqrt[3]{d^2}$ Unsupported Length of Shafts. or shafts supporting their own weight only; $L = 50\ \sqrt[3]{d^2}$ for shafts arrying pulleys, gearing, etc., where L = length of shaft between enter of bearings, in inches; and d = diameter of shaft, in inches.

82.—Line shafts connecting machinery at the center to that at the nds shall run at fairly high speed. The speed reduction shall be made n the machinery near the end.

83.—In designing circular shafting, trunnions, and axles, the reatest unit fiber stress in tension or compression due to bending hall be calculated by the following formula:

$$f = \frac{32}{\pi d^3}\left(\frac{3}{8}M + \frac{5}{8}\sqrt{M^2 + T^2}\right).$$

84.—The maximum unit shear shall be calculated by the following Formulas for Shafts. ormula:

$$S = \frac{16}{\pi d^3}\sqrt{M^2 + T^2}.$$

85.—In these formulas, f = unit fiber stress in tension or com- ression; S = unit shear; d = diameter of shaft; M = the simple ending moment; and T = the simple twisting moment.

86.—If a shaft, trunnion, or axle has one key-way cut, f and S shall Effect of Key-Ways in Shafts. e increased by one-sixth; if two key-ways are cut, increase by one- ourth. If the shaft, etc., is enlarged through the hub, this does not pply.

87.—In calculating the bending moment on shafts, trunnions, and Distance Between Shaft Supports. ournals, the distance from center to center of bearings shall be taken.

88.—Gear teeth shall be of the involute type, with an angle of Style of Gear Teeth. bliquity of 20 degrees. The roots below the clearance line shall be lleted.

89.—The width of the teeth may be as great as four times the itch, but not more, except for wheels running at a very high velocity, s in motors where abrasion is to be considered.

Strength of Beveled Gear Teeth.

90.—In estimating the strength of teeth in bevel wheels, the pitch at the middle section shall be taken.

Pitch Circle.

91.—For the purpose of setting gear teeth accurately in the field erection, the pitch circle shall be scribed on the ends of the teeth.

Worm Gearing.

92.—Worm gearing, for transmitting power, shall have an angle of thread not less than 20 degrees. The worm shall run in oil. A bronze or brass collar shall be used at the end of the worm and at the end of the wheel axle, to take care of the end thrust. The wheel shall be of bronze. If a nut engages the worm, the nut shall be of bronze.

92a.—Worm wheels shall have not less than twenty-eight teeth.

93.—Worms which are to be used for actuating signals, indicators, or other minor parts may have an angle of thread less than 20 degrees.

93a.—Safety guards shall be provided around gears and other moving parts where it is necessary for workmen to be while the machinery is in motion.

Counterbalancing, Operating Ropes, and Attachments.

Wire Ropes and Cables.

94.—Wire rope shall be made by some manufacturer approved by the Engineer.

95.—The counterbalance ropes shall be of plow-steel wire, and shall consist of six strands, of nineteen wires each, laid around a hemp center.

96.—Ropes shall be laid up in the best manner, and shall be thoroughly soaked in an approved lubricant during the process of manufacture.

97.—The counterbalance ropes shall be made from wire which has been tested in the presence of an inspector designated by the Engineer, and which, for sizes from 0.076 to 0.150 in. in diameter (the limiting values used in counterbalance ropes), exhibits the following physical properties:

a.—The tensile strength shall be not less than 225 000 lb. per sq. in. for wire from 0.150 to 0.126 in., nor less than 230 000 lb. for wire from 0.125 to 0.101 in. in diameter; nor less than 235 000 lb. for wire from 0.100 to 0.076 in. in diameter.

b.—The total ultimate elongation, measured on a piece 12 in. long, shall be not less than 2.4 per cent.

c.—The number of times a piece 6 in. long can be twisted around its longitudinal axis without rupture shall be not less than 1.4 divided by the diameter, in inches.

d.—The number of times the wire can be bent 90° alternately to the right and to the left, over a radius equal to twice its diameter, without fracture, shall be not less than six. This test shall be made in a mechanical bender constructed so that the wire actually conforms to the radius of the jaws and is subjected to as little tensile stress as possible.

98.—The rope shall be made in one piece, if possible. Its breaking strength, as determined by the test described in Paragraph 101, shall be not less than

Ultimate Strength of Cables.

4 900	lb.	if	$\frac{1}{4}$	in.	in diameter.
11 800	"	"	$\frac{3}{8}$	"	" "
20 600	"	"	$\frac{1}{2}$	"	" "
32 400	"	"	$\frac{5}{8}$	"	" "
45 000	"	"	$\frac{3}{4}$	"	" "
70 200	"	"	$\frac{7}{8}$	"	" "
79 200	"	"	1	"	" "
100 800	"	"	$1\frac{1}{8}$	"	" "
120 600	"	"	$1\frac{1}{4}$	"	" "
148 000	"	"	$1\frac{3}{8}$	"	" "
173 000	"	"	$1\frac{1}{2}$	"	" "
200 000	"	"	$1\frac{5}{8}$	"	" "
230 000	"	"	$1\frac{3}{4}$	"	" "
264 000	"	"	$1\frac{7}{8}$	"	" "
297 000	"	"	2	"	" "
325 000	"	"	$2\frac{1}{8}$	"	" "
374 000	"	"	$2\frac{1}{4}$	"	" "
465 000	"	"	$2\frac{1}{2}$	"	" "

99.—In case the breaking strength of the rope falls below the values cited above, the entire length from which the test pieces were taken shall be replaced by the manufacturer with a new length, the strength and physical qualities of which come up to the specifications.

100.—Sockets used in connection with this rope shall be forged, without welds, from solid steel. In every case the dimensions shall be such that no part under tension shall be loaded higher than 65 000 lb. per sq. in. when the rope is stressed to its ultimate strength, as named above. The sockets must be attached to the rope by a method which is absolutely reliable and will not permit the rope to slip in its attachment to the socket.

101.—In order to show the strength of the rope and fastenings, a number of test pieces, not more than 10% of the total number of finished lengths which will be ultimately made, nor less than two from each original long length, and not more than 12 ft. long, shall be cut, and shall have sockets, selected at random from those which are to be used in filling the order, attached to each end. These test pieces are to be stressed to destruction in a suitable testing machine. Under this stress the rope must develop the ultimate strength given in Paragraph 98.

102.—The sockets must be fastened to the rope so that there is no slipping of the rope in the basket. If slipping should occur, then the method must be changed until one is found whereby slipping can

be entirely avoided. The sockets themselves shall be stronger than the rope with which they are used; if one should break during the test, then two others shall be selected and attached to another piece of rope and the test repeated; and this process shall be continued until the inspector is satisfied of their reliability, in which case the lot shall be accepted. If, however, 10% or more of all the sockets tested break at a load less than the minimum ultimate strength of the rope given in Paragraph 98, then the entire lot shall be rejected and new ones shall be made of stronger material.

Length of Rope.

103.—The length of each rope, from inside of bearing to inside of bearing of sockets, shall be determined, and a metal tag having the said length stamped thereon shall be securely attached to the rope.

104.—The purchaser reserves the right to test each wire rope connection, after its attachment is made, up to one-half of the ultimate strength of the rope, and, if it shows the least sign of weakness, it shall be rejected and replaced.

Facilities for Testing Rope.

105.—The manufacturer shall provide proper facilities for making the tests, and shall make at his own expense all the tests required. Tests shall be made in the presence of an inspector who represents and is paid by the Engineer.

Shipment of Rope in Coils.

106.—Ropes shall be shipped in coils the minimum diameter of which is at least thirty times that of the ropes, and they shall be uncoiled for use by revolving the coil, not by pulling the rope away from the stationary coil.

Equalizing Levers.

107.—The equalizing levers connecting the ropes to the counterweights and their pins more than $3\frac{1}{2}$ in. in diameter shall be of forged steel; pins $3\frac{1}{2}$ in. in diameter or less shall be of rolled machinery steel. The levers shall be neatly finished, and shall conform to the dimensions shown on the drawings.

Workmanship.

108.—For the parts of the operating machinery of movable bridges which are usually exposed to the weather, the finish shall be confined to the bearing, rotating, and sliding surfaces, and wherever it is required to produce accurate fits and precise dimensions.

109.—Castings shall be cleaned, and seams and other blemishes removed.

110.—Drainage holes, not less than $\frac{3}{4}$ in. in diameter, shall be drilled in places where water is likely to collect.

Play of Unfinished Bolts.

111.—Unfinished bolts may have a play of $\frac{1}{16}$ in. in the bolt holes. Turned bolts must have the diameter of the shank at least $\frac{1}{16}$ in. larger than the diameter of the threaded portion, and must have a driving fit in the bolt hole.

Racks and Contact Surfaces.

112.—The backs of racks and contact surfaces shall be planed.

Tread-Plates.

113.—The top and bottom of the tread-plates and contact surfaces in rolling bridges shall be planed to fit. A full bearing must be made.

114.—The periphery and the ends of teeth which mesh with a shrouded pinion shall be planed, and the pitch line scribed thereon.

115a.—The joints between the caps and bases of journal and trunnion bearings shall be planed. The ends of the bases and surfaces in contact with the supports shall be planed. Bolt holes for holding the cap to the base and for holding the base to its support shall be drilled.

Finishing of Trunnions, etc.

115.—Journals and trunnions shall be turned with a fillet at each end and at points where the section changes. Trunnions and journals 8 in. and more in diameter shall have a hole, $1\frac{1}{2}$ in. in diameter, bored through on the longitudinal axis. Journals, trunnions, and bushings must be polished after being turned. The use of a cutter which trembles or chatters will not be allowed.

Grooves.

116.—The grooves in the surfaces of trunnions or similar large bearings shall be machine cut. Chipping and filing will be allowed only for removing small inequalities. The grooves shall be smooth, especially the rounded corners.

Hubs.

117.—Hubs of wheels, pulleys, couplings, etc., shall be bored to fit close on the shaft axle. If the hub performs the function of a collar, the end next to the bearing shall be faced. Holes in hubs of toothed gear wheels shall be concentric with the pitch circle.

Cut Gears, etc.

118.—The periphery of gear wheels shall be turned. Gear wheels which are part of the train which actuates the moving span, or the bridge-locks, or the rail-locks, shall be cut. Machine-moulded teeth may be used for actuating signals or small parts.

Beveled Gears.

119.—Beveled gears shall be cut. The cutting shall be done by a planer having a rectilinear motion to and from the apex of the cone. Rotating milling cutters shall not be used.

Grooves in Sheaves.

120.—The grooves in the circumference of sheaves carrying wire ropes shall be turned to a radius which will fit the rope. This is to be done after the sheave is completely assembled and permanently riveted up.

121.—At the juncture of the shrouding and teeth in pinions, cleaning, chipping, or other means shall be used to insure the meshing of the pinion teeth and rack teeth.

122.—Threads on worms, and the teeth of worm wheels shall be cut and shall fit accurately. Point contact shall be avoided.

123.—Any two surfaces which slide, roll, or bear on each other shall be planed.

Assembling of Machinery.

124.—Machinery parts shall be assembled on the supporting members in the shop, and shall be aligned and fitted, with holes in the supports drilled, and with the members in correct relative position. The members shall be match-marked, both to the supports and to each

other, and re-erected in the same relative position; or, if not assembled in the shop, connecting holes in the supports shall be drilled in the field.

Holes for Sheaves for Vertical Lift Bridges.

125.—The holes in the girders and columns for the bolts connecting the main sheave bearings to their supporting girders shall be drilled from the solid through cast-iron or steel templets on which the bearings were set and accurately lined when the holes in the bearing were bored. The bolt holes and the bolts shall be turned to the same diameter and the bolts driven to place without injury to them, the bearings, or the girders or columns.

Shop Test on Trunnions.

126.—If trunnions rotate in fixed pedestal bearings, such as the sheave trunnions in vertical lift bridges or similar bearings, the pedestals shall be firmly mounted in the shop, the trunnions placed therein and covers bolted; the whole, when assembled, shall simulate the assemblage in the field as nearly as practicable. The maximum torque in inch-pounds required to rotate the trunnion shall be $\frac{W\,r}{10}$, where W equals the weight of the trunnion, in pounds, and r equals the radius of the trunnion, in inches. If large structural parts rotate about the axis of the trunnion, the trunnion shall be inserted in its bushing in the structural part and rotated. If the shop position of the structural part is flat, which is the usual case, the axis of the trunnion will be vertical, and there will be no load on the bearing; in this case the maximum torque required to rotate the trunnion shall be $\frac{W\,r}{50}$. At least four complete rotations of the trunnion must be made. If any grinding or hard turning is found, it must be remedied. These trunnion tests shall be made in the presence of the Railroad Company's inspector, and with such apparatus as will readily determine the torque.

Facing of Couplings.

127.—Faces of flange and split muff couplings shall be planed to fit. The couplings shall be keyed to the shaft.

128.—A special effort to secure good workmanship on keys and key-ways shall be made.

Coating of Surfaces.

129.—Machined surfaces shall have a coating of white lead applied to them.

130.—Machinery which is of the regular standard manufactured type, such as steam, gasoline, electric motors, pumps, air compressors, etc., shall be guaranteed by the manufacturer as to efficiency, and shall be subject to the approval of the Engineer. Motors shall be tested to prove that they fulfill the specified requirements and develop the desired speed, power, and torque.

Brake Test of Motors.

131.—The rating of a motor shall be the horse-power determined by the brake test.

132.—The electric equipment shall conform to the Standardization Rules of the American Institute of Electrical Engineers, as approved June 21st, 1907. (See "Standard Hand Book for Electrical Engineers," 3d Edition, Sect. 19.) A. I. E. E. Rules.

133.—The unit stresses per square inch, to be used for parts in which main stresses are not increased by impact, shall be as follows:

STRESSES IN ONE DIRECTION, IN POUNDS PER SQUARE INCH.

Material.	Tension.	Compression.	Fixed Bearing.	Shear.
Machinery steel.....	9 400	$9\,400 - 40\,\frac{l}{r}$	11 000	6 200
Structural steel.....	8 500	$8\,500 - 36\,\frac{l}{r}$	.....	5 600
Steel castings.....	7 000	$8\,000 - 35\,\frac{l}{r}$	.....	5 000
Phosphor-bronze.....	6 600	.....	.....	4 600
Cast iron	3 000	8 000	.....	3 000
Shear on keys....	.....	4 900 lb.		
Bearing on keys.	.....	8 800 "		

134.—The maximum unit tension in plow-steel cables shall be one-sixth of the ultimate. The maximum unit tension is equal to the direct unit stress plus the extreme fiber unit stress in the individual wire due to bending over the sheave.

135.—For stresses which are reversed at the rate of five or more times per minute, use one-half of the above unit stresses. Reversal of Stresses.

136.—If wire rope is bent over a sheave, the bending stress and permissible load on the rope shall be calculated as follows:

Let P = the total pull or permissible load, on the rope, in pounds;
K = extreme unit fiber stress in the greatest individual wire;
E = modulus of elasticity = 28 500 000;
a = cross-sectional area of rope, in square inches;
d = diameter of thickest wire, in inches;
D = diameter of sheave to center of rope, in inches;
S = greatest unit tension allowable;
α = angle of helical wire with axis of strand;
β = angle of helical strand with axis of rope;
c = diameter of rope.

$$\text{Then } K = \frac{Ed \cos.^2 \alpha \cos.^2 \beta}{D} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

$$P = a\left(S - \frac{Ed \cos.^2 \alpha \cos.^2 \beta}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots (2)$$

For rope having six strands of nineteen equal wires each,

$$P = a\left(S - \frac{1\,800\,000\,c}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

because $cos.^2\,\alpha\ cos.^2\,\beta = 0.95$, $d = \frac{c}{15}$.

137.—For haulage rope, six strands of seven wires each, take $d = \frac{c}{9}$.

138.—If a rope is in contact with a sheave over a small arc, the actual radius of curvature may be greater than that of the sheave. (Fig. 2.)

Let R = the actual radius of curvature;
θ = the angle between the directions of the rope;
W = pull on individual wire, equal to P divided by the number of wires if all wires are of equal diameter.

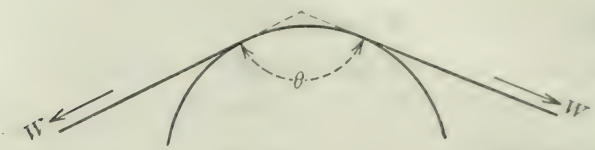


Fig. 2.

Then $$R = \frac{d^2}{4\ cos.\ \frac{\theta}{2}}\sqrt{\frac{E}{W}}$$

139.—If R is greater than the radius of the sheaves, $2R$ should be used in place of D in Formulas 1, 2, and 3. The formula is only valid for θ between 110 and 180 degrees.

Strength of Gear Teeth.

140.—The strength of cut gear teeth shall conform to the following formula, one tooth only taking pressure:

$$P = fpb\left(0.154 - \frac{0.912}{n}\right)\frac{600}{600 + V}.$$ in which

P = pressure on tooth, in pounds;
f = permissible unit stress = 17 000 lb.;
p = pitch, in inches;
b = face or breadth of tooth, in inches;
n = number of teeth in gear;
V = velocity on pitch circle, in feet per minute.

141.—The strength of machine-moulded teeth shall be calculated by the foregoing formula, taking $f = 15\,000$ lb.

142.—The strength of shrouded teeth shall be computed as for uncut teeth, the purpose of the shrouding being to provide for future wear of pinions.

143.—The foregoing formula is for involute teeth having an angle of obliquity equal to 20 degrees.

144.—The pressure, in pounds per linear inch, on rollers at rest shall be, for rolled and cast steel, 600 d, where d equals the diameter of the roller, in inches. Pressure on Rollers.

Unit Stresses for Bearing on Rotating and Sliding Surfaces.

145.—The maximum bearing values for rotating and sliding surfaces, in pounds per square inch, shall be as follows:

For bearings on which the speed is slow and intermittent:

	Pounds per square inch.
146.—Pivots for swing bridges: Hardened tool steel on special phosphor-bronze	3 000
147.—Trunnion bearings on bascule bridges: Axle steel on phosphor-bronze, average	1 500
and never greater than 1 700 lb. for maximum bearing for any position of the bridge.	
148.—Wedges: Cast steel on cast steel or structural steel	500
149.—Screws which transmit motion on projected area of thread	200
150.—For ordinary cases, parts moving at moderate speeds:	
Hardened steel on hardened steel	2 000
Hardened steel on bronze	1 500
Tool steel (not hardened) on bronze	900
Structural steel on bronze	600
Cast iron on structural steel	400
Cast iron on cast iron	400
On cross-head slides, speed not exceeding 600 ft. per min.	50

151.—In order to prevent heating and seizing at higher speeds, the pressure on pivots or foot-step bearings for vertical shafts and journals shall not exceed:

$$\text{On pivots} \ldots\ldots p = \frac{40\,000}{n\ d} \text{ per square inch.}$$

$$\text{On journals} \ldots\ldots p = \frac{300\,000}{n\ d} \text{ per square inch.}$$

Where n = number of revolutions per minute,
and d = diameter of journal or pivot, in inches.

152.—For crank pins and similar joints with alternating motion, the limiting bearing values given in the above formula may be doubled.

Stresses on Rollers.

153.—The permissible pressures, in pounds per linear inch of roller in motion, shall be as follows:

For cast iron	$p = 200d$
For steel castings	$p = 400d$
For axle steel	$p = 500d$
For tool steel	$p = 800d$
For hardened tool steel	$p = 1\,000d$

Where p = pressure per linear inch of roller,
and d = diameter of roller, in inches.

154.—The foregoing values are for rollers and bearing surfaces of the same material; if rollers and bearing surfaces are of different materials, the lower value shall be used.

Motors.

155.—The kind of motor best adapted to any particular case depends on local conditions, and should be left to the judgment of the Engineer.

Mechanical Power.

156.—If the bridge is operated by mechanical power, the motor shall be of ample capacity to move or turn the bridge at the required speed. All machinery parts shall be designed with sufficient strength to resist the greatest pressure which can be exerted by the motor. No matter what mechanical power is used, all bridges shall also be provided with hand-power operating machinery.

Friction Brakes.

157.—Friction brakes, to be operated by hand or foot, shall be provided where the motor is located in the operator's house. They shall be attached to the secondary shaft of the motors which connect to the moving gear, and shall have sufficient capacity to stop or hold the moving span in any position, under all conditions.

Operator's House.

158.—If mechanical power of any kind is to be used for operating a movable bridge, a suitable house shall be provided for the operator. The house shall be of such dimensions as required for the purpose for which it is to be used. It shall be placed in a position where the operator can observe the signals and see the approaching vessels and trains, and with enough windows of sufficient size, so that this view will not be obstructed. If the operator's house is above or below the floor of the bridge, suitable steel or iron stairs with railings shall be provided to lead from the floor of the bridge to the floor of the operating house. The house shall be of fire-proof construction, consisting of a steel frame, steel floor-joists and a fire-proof floor. If the house contains motors and machinery, the floor shall preferably consist of steel plates, but, if the motors are located elsewhere, the floor between the joists may be of concrete construction. The sides and roof shall be of metal, concrete, or any other non-combustible material. The hand-rail for stairways and other places shall be of $1\frac{1}{2}$-in. gas pipe.

Heating of Operator's House.

159.—Whenever the climatic conditions require it, provision shall be made for heating the operator's house. If steam power is used, the house shall be heated by a steam coil or radiator fed from the boiler. If electric power is used, the heat may be supplied by electricity. If gasoline is used, or any other power which cannot be utilized for heating, a coal, wood, petroleum, or gas stove, as directed by the Engineer, shall be provided.

Steam Engine.

160.—If a steam engine is used, it shall consist of a double-cylinder, reversing engine, the piston speed of which shall not exceed 200 ft. per min.; it shall develop the desired power and speed with a steam pressure of 50 lb. per sq. in. The engine shall be connected to the operating machinery by an approved friction clutch, arranged so that the moving and locking machinery can be operated alternately or stopped without stopping the engine.

Steam Separator.

161.—In the steam supply pipe, and close to the steam chest, shall be placed a steam separator. This separator, under test with quality of steam as low as 66%, shall show an average efficiency of 85% in five tests.

Boilers.

162.—The steam shall be generated by one or two upright, tubular boilers, each of which shall have twice the capacity of the engine. The boilers shall be designed for a steam pressure of 150 lb. per sq. in., and shall be adapted to the kind of fuel specified by the Engineer; they shall be of open-hearth steel in accordance with the specifications for boiler plates, Paragraphs 246 to 251, inclusive. They shall be encased in asbestos and covered with Russia iron.

163.—The boilers shall also be in accordance with the specifications of the Mechanical Department of the Railway Company, and shall conform to the civil laws.

Flues of Boilers.

164.—Vertical boilers shall have submerged flues at the top.

Horse-Power of Boilers.

165.—The total horse-power of the boilers shall be twice that of the engine, and shall be computed by the following rule: Calculate the inside area of the tubes, the area of tube sheet next to the fire, and the sides of the fire-box where this is in contact with the fire. Take the sum of these areas in square feet and divide by fifteen. The intention is to allow 15 sq. ft. of heating surface per horse-power. At least $\frac{1}{3}$ sq. ft. of grate surface shall be provided per horse-power.

Equipment of Engine-Room.

166.—The engine-room shall be provided with a steel water tank of sufficient capacity; a duplex, steam feed-pump; and an injector for each boiler, with necessary pipes and connections for feeding boilers separately or together; steam water-lifters with necessary strainers, flexible hose, and piping to lift the water from the river into the tank; a coal hoist and a steel coal-bin of sufficient capacity. The engine-room shall be provided with a suitable indicator for recording the positions of the moving span in turning and locking. A work-bench with

a full set of machinist's tools shall be provided, such as a vise, wrenches, chisels, hammers, files, oilers, oil-cans, and oil-tank.

Whistle.

167.—A whistle having a bell 5 in. in diameter and 12 in. long, shall be installed complete. If operated by air, the compressor and air tank shall conform to the following specifications: The compressor shall be motor driven, the motor and compressor being on one frame, and geared. All working parts shall be completely enclosed, and self-lubricating. The compressor shall have a piston displacement of from 25 to 30 cu. ft. per min. when working against a tank pressure of 90 lb. per sq. in. The compressor shall be provided with strainer, and automatic governor and switch, in order that the compressor may start and stop automatically at any predetermined tank pressure. The air receiving tank shall be 36 in. in diameter and 8 ft. long, or of equal capacity. The tank shall be galvanized, and good for a working pressure of 100 lb. per sq. in. It shall be provided with pressure gauge and pigtail, pop-valves and drain cock, and have standard flanges bushed for 1½-in. pipe. The Contractor shall furnish all pipe, pipe fittings, and valves, and all shall withstand a working pressure of 100 lb. per sq. in.

Gasoline Motor.

168.—If a gasoline motor or other internal-combustion motor is used, a low-speed engine of the most substantial kind shall be selected, the maximum piston speed of which shall not exceed 350 ft. per min. The engine shall have a reversing gear provided with approved friction clutches, to be operated by a hand-wheel. The countershaft connecting the engine with the operating machinery shall be provided with disengaging couplings, arranged so that the moving and locking machinery can be operated alternately and in either direction without stopping the engine. Motors of 10 h.p. and more shall be started by compressed air. The engine-room shall be provided with a water tank of sufficient capacity. The gasoline tank shall be located outside of the engine-house. The engine-room shall be provided with indicators for recording the positions of the moving span, and lifting and locking apparatus. A work-bench with a full set of machinist's tools, etc., shall be provided, the same as specified for steam engines.

Electric Motors.

169.—Electric motors and generators, if for direct current, shall be of the railway series, interpole type, water-proof, with slotted-drum armature, and form-wound armature coils. They shall be a standard commercial type in common use.

170.—The coils shall be impregnated.

171.—Motors, generators, automatic circuit breakers, solenoids, brakes, and other electric mechanism shall be tested at the factory by the manufacturer in the presence of the Railway Company's inspector.

Testing of Motor.

172.—The rating of a direct-current motor is the horse-power output at the armature shaft which gives a rise of temperature above the surrounding air (referred to a room temperature of 25° cent.) not exceeding

90° cent. at the commutator and 75° cent. at any other part after one hour's continuous run at its rated voltage (and frequency in the case of an alternating-current motor) on a stand with the motor covers removed and with natural ventilation. The rise in temperature is to be determined by thermometer, but the resistance of no electric circuit in the motor shall increase more than 40% during the test.

172a.—Alternating-current motors shall show, in a run for heat test, the following maximum temperature rises above 25° cent. for the surrounding room: For ½-hour run under nominal load, 40° cent.; for 3-min. run under 150% nominal load, 55° cent.

173.—Direct-current motors shall be capable of carrying a load of 200% for 3 min. with the same temperature rise and momentarily of 400% without injury, starting cold in each instance.

Torque of Motor.

174.—The motors under test shall develop the required horse-power and torque at the armature shaft. Characteristic curves showing the results of the test shall be furnished by the manufacturer.

175.—The motor frame shall have two bearings for the countershaft and shall have a forged-steel cut pinion, out of one piece, keyed to the end of the armature shaft and secured by a lock-nut.

175a.—If the motor is enclosed in a case, as mill motors are, small openings of sufficient size shall be provided in the case for the inspection, removal, and replacing of brushes.

176.—One cast-steel cut gear, bored and key-seated for attachment to the countershaft, shall be furnished with the motor. The gear and pinion shall be covered by a sheet-steel or malleable-iron split gear case, supported by the motor frame and completely covering the gear and pinion. An opening, with a hinged cover, shall be provided in the gear case for inspection and oiling. The gear ratio shall be such that the full speed of the countershaft will not be more than 125 rev. per min.

Spare Motor Parts.

177.—For each size of motor furnished, the Contractor shall supply the following spare parts: One armature, one field coil, one pinion, one gear, and one set of brushes. These parts shall be finished and fitted in such a manner as to admit of being installed in their respective places without further fitting or adjustment.

Mounting Motors.

178.—The motors shall be mounted in such a manner as to admit of easy access for inspection and repairs; they shall be supported securely by brackets or suitable foundations.

179.—If the machinery and motors are on the moving span, they shall be capable of being operated satisfactorily in any position of the span.

Controllers.

180.—The controllers for motors shall be located in the operating-house. The controllers shall be of the reversing drum type, with magnetic blow-out, and shall be capable of varying and maintaining

the speed of the motors throughout the entire range desired, without injurious sparking, and without shock due to sudden variation in speed. The controllers shall be capable of doing their work for the usual loads, and excess loads, that may come upon the motors, with a temperature rise not exceeding that specified for the motors.

181.—One controller, with the necessary resistances, shall be furnished for controlling the operation of each main operating motor. They shall be connected so that the motors may be operated together.

Type of Controllers.

182.—The controllers for direct-current motors shall be of the series-parallel type; or of the type in which the field is varied, as may be done for the interpole type of motor.

Controllers, Where Needed.

183.—One controller shall be furnished for the operation of the rail-locks, and one for bridge-locks. These controllers shall be designed so that the operation of any motor can be cut out by pulling a switch on the switch-board, without affecting the operation of any of the other motors.

Automatic Cut-Offs.

184.—An automatic cut-off or short-circuiting device shall be provided which will throw out the circuit breakers, cut off the current from the operating motors and set their brakes when the bridge is 5° from its open position, and its closed position. Spring switches shall be provided which, if closed and held closed, will put the cut-offs out of commission and thus enable the bridge tender fully to close or open the bridge.

185.—The end-lock motor shall be stopped and its brake set automatically at each end of its travel.

Resistances.

186.—Resistances shall be of the cast-grid type, and of such capacity that the motor can be operated continuously at any point of the controller when developing full-load torque, or for 10 min. when developing 50% over-load torque, without sufficient rise in temperature of the resistance to cause deterioration of any part. The resistances shall be mounted so as to admit of free ventilation and be without injurious vibration.

Electric Brakes.

187.—The main operating motors, rail-lock motors, and bridge-lock motors shall be provided with approved post brakes which are held in set position by a spring with such force as to overcome not less than 50% of the maximum torque required. The friction surfaces are to be of materials not affected by moisture. The brakes are to be released by solenoids of ample power and heating capacity whenever the motors are taking current, and are to be automatically set whenever the current fails or is cut off from the motors. Weather-proof motors shall be provided with weather-proof solenoids. Brakes shall be provided with a foot-switch release for coasting purposes. Means shall be provided for mechanically releasing the brakes when the bridge is to be operated by hand or other equipment.

Emergency Brakes.

188.—An additional emergency brake shall be provided and applied to the main operating machinery. This shall be released by means of a motor-operated mechanism furnished by the Electrical Contractor, which shall hold the brake in release as long as the current is applied to the brake motor. Cutting off the current from this brake motor, or any failure of current, will result in the instantaneous application of the brake. This brake will be normally set, but will be released by the operator before starting the bridge, and be held in release during the entire operation unless an emergency condition arises requiring brake power in excess of that offered by the motor brakes, in which case it may be instantly applied by the operator. After the bridge has been closed and traffic has been resumed, this brake will again be applied. This portion of the equipment shall be designed so that it will not be injured if left in release indefinitely. Proper means shall be provided for releasing the brake mechanically when the bridge is to be operated by hand or emergency-power equipment.

189.—The emergency brake motor circuit is to be independent of the general interlocking system, and there shall be a mechanical interlocking device which will prevent the main leaf motors and the emergency brake from being used one against the other.

190.—The emergency-brake switch shall be attached to the controller stand within easy reach of the operator and proper labels shall be placed back of the switch handle to indicate "Set" and "Released" positions of the brake.

Current Supply.

191.—Unless the current supply is taken from more than one source, it shall be conducted to the switch-board in two independent conductors, one for the supply, and one for the return current.

Submarine Cables.

192.—Submarine cables, if needed, will be furnished and laid by the Railroad Company.

193.—The wiring from the collector rings for the electrical equipment of the bridge shall be furnished by the Contractor.

Qualities of Wire and Insulation.

194.—The quality of all wires and insulation shall conform to the specifications of the Railway Signal Association, as revised and adopted in October, 1911, and contained in Volume 8 of the *Proceedings* of that Association.

Conduits and Minimum Size of Wire.

195.—If wires are to be placed in conduits, the conduits shall be of ample size, sherardized, and loricated on the inside. No wire less than No. 12, B. & S. gauge, shall be used.

Condulets and Factory Ells.

196.—Conduits shall be of sufficient size to allow the wires to be easily drawn in. No joints are to be made inside of a conduit. Condulets and factory ells shall be used. Condulets, ells, and conduits shall be sherardized, and loricated inside.

Wiring, etc., to Conform to Codes.

197.—The wiring, motor installation, and the whole electric equipment must conform to the underwriter's code, and to the city code, if the bridge is subject to city authority.

Fuses.

198.—Enclosed fuses shall be used.

Minimum Stranded Wire.

199.—No wire smaller than No. 10, B. & S. gauge, stranded wire, shall be used.

Wires to be Tagged.

200.—Wires when installed shall be permanently tagged and numbered so that any wire can be traced from the switch-board to the motors, and to the source of power.

Ground Connections. Quick-Break Switch and Switch-Board.

201.—Ground connections of ample area shall be provided.

202.—A switch, of the quick-break type, shall be provided for each supply wire. Each motor circuit and each light, signal, indicator, or other circuit shall be provided with switches which are approved by the Railway Company's Engineer. The switches shall be mounted on an enameled slate panel switch-board (not less than 1¼ in. thick, and free from metallic veins, or flaws) in the operator's house. The switch-board shall be large enough to carry the meters, switches, cut-outs, fuses, etc. Switches, cut-outs, buttons, etc., shall be provided with plates designating their use.

Automatic Circuit Breaker.

203.—An automatic circuit breaker shall be placed on the switch-board in the operating motor circuit of the bridge. Each line to the motor, each line to the electric brakes, and each lighting, signal, indicator, or other circuit, shall be protected by enclosed fuses.

204.—Any circuit whatsoever shall be protected by fuses, circuit breakers, or equivalent devices, which will insure the excessive current being cut off before any parts are damaged.

Lightning Arrester.

205.—The feeders shall be protected by a pole-switch fuse and lightning arrester mounted on a non-combustible and non-absorbent insulating base.

206.—Lightning arresters shall be placed as near as practicable to the parts to be protected, and away from combustible material. A No. 4, B. & S. gauge, wire should be used for the connection; this wire should run in a straight line to a ground plate, and not be connected to any structural parts. To avoid inductive resistances, the wire should not run through a conduit. If a choke-coil is used, it should be thoroughly insulated from the ground and other conductors.

Short Circuiting.

207.—The connections of parts in contact with track shall be such as to allow no short circuiting of track signals.

Protection of Electric Contacts.

208.—Electric contacts shall be protected from the weather or accumulations of dirt.

Housing of Motors.

209.—Motors must be housed in weather-proof metal housing. This housing must be large enough to allow the inspection and oiling of the motor. It must be readily removable so that access to the motor may be obtained. No metal in this housing shall be less than No. 16, U. S. Standard, gauge; it shall be galvanized.

Housing of Solenoids, etc.

210.—Solenoids and electrically-operated brakes shall be housed.

Indicator Lights.

211.—The Contractor shall provide and install electric light indicators for the purpose of showing the operator the various positions

of the bridge, especially the fully open, entirely closed, nearly open, and nearly closed positions of the bridge, and the fully open and fully closed positions of the rail-lock and bridge-locks.

Volt Meter, etc.

212.—A volt meter, ammeter, and watt meter shall be provided on the switch-board. The use of external multiple shunts will not be permitted.

Ground Detector.

213.—The switch-board shall be furnished with one 2-c. p. lamp for detecting ground, and a 2-c. p. lamp for illuminating the ammeter and volt meter scales.

Lamps for Lighting.

214.—In the operator's house shall be placed ten 16-c. p. lights, and additional lights about the machinery and such other lights as the Engineer may direct. For all lights in the house above ten in number, the Railroad Company will pay the regular market price or furnish them to the Contractor.

215.—Lights of 16-c. p. shall be placed outside at the head and foot of stairways or similar paths. All lights in the house shall have tungsten filaments.

Channel Lights.

216.—The Contractor shall furnish warning and channel lights and signals, in accordance with the U. S. Government requirements, or other harbor requirements.

Alternating Current Motors.

217.—Alternating motors shall be of the three-phase induction type with slip-rings, rotor-wound, 25 cycles and 220 voltage, unless otherwise specified. The resistances for varying the speed shall be in series with the rotor circuit, and shall be such as to affect evenly all three phases. Motors of 5 h.p. or less may be of the squirrel-cage type.

218.—Alternating-current motors shall be of the crane type, and be of rugged construction. The motors shall be capable of exerting momentarily a maximum torque equal to at least two and a quarter times the greatest torque shown by the torque curves for the bridge and operating machinery, even if it requires a motor having a nominal horse-power greater than that which may be found from the torque curves (Paragraph 31).

Control of Motors.

219.—The control of motors shall be electrically interlocked with each other and with the signal system, and the bridge shall be controlled in such a way that the end-locks or wedges cannot be released until the signals have gone to the danger position and the derails are set, or the bridge motor started until the end-locks and wedges have actually been released. In closing the bridge, the control shall be such as to make it impossible for the operator to move the end-locks or wedges until the bridge has been completely closed, or to set the signals at safety until the bridge has been closed and the end-locks and wedges are in place.

Railway Signal System.

220.—The company will furnish and install the railway signal system, also the master lever and all necessary devices controlling the interlock between this signal system and the bridge as a whole. The

Contractor shall furnish and install the necessary devices for interlocking the various parts of the bridge with each other and for connection to the Company's master lever.

Specifications for Special Metals Used for Machinery Parts.

Qualities of Machinery Steel.

221.—Steel for castings may be made by the open-hearth or crucible process.

222.—All castings shall be annealed unless otherwise specified.

223.—Phosphorus ..... 0.05% maximum.
Sulphur ..... 0.05% maximum.

224.—Minimum physical qualities, as determined on a standard test specimen, of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch.....	70 000
Elongation: percentage in 2 in.....	18
Contraction of area: percentage.....	25

225.—A test to destruction may be substituted for the tensile test, in the case of small or unimportant castings, by selecting three castings from a lot. This test shall show the material to be ductile, free from injurious defects, and suitable for the purpose intended. A lot shall consist of all castings from the same melt or blow, annealed in the same furnace charge.

Flaws in Castings

226.—Castings shall be true to pattern, free from blemishes, flaws, or shrinkage cracks. When the bearing surface of any steel casting is finished there shall be no blow-holes visible, exceeding 1 in. in any direction, nor exceeding ½ sq. in. in area. The length of blow-holes cut by any straight line laid in any direction shall never exceed 1 in. in any 1 ft.

Blow-Holes in Gear Wheels.

227.—No blow-hole exceeding one-half the above dimension and area will be allowed in any gear tooth, or in the rim at the root of the teeth.

Electric Welding.

228.—The correction of defects in castings, by welding electrically by thermit or by similar processes, will not be allowed.

Testing of Large Castings.

229.—Large castings shall be suspended and hammered all over. No cracks, flaws, defects, or weakness shall appear after such treatment.

230.—A specimen (1 in. by ½ in.) shall bend, cold, around a diameter of 1 in., through an angle of 90°, without fracture on the outside of the bent portion.

231.—The number of standard test specimens shall depend on the character and importance of the castings. A test piece shall be cut, cold, from a coupon to be moulded and cast on some portion of one or more castings from each melt or blow, or from the sink-heads (in case heads of sufficient size are used). The coupon or sink-head must receive the same treatment as the casting or castings, before the specimen is cut out, and before the coupon or sink-head is removed from the casting.

232.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred

by the inspector, shall be used to determine whether or not the steel is within the limits in phosphorus and sulphur specified in Paragraph 223 concerning chemical properties.

Steel Forgings.

Qualities of Steel Forgings

233.—Steel forgings may be made by the open-hearth or crucible process.

234.—Phosphorus ..... 0.04% maximum.
Sulphur ..... 0.05% maximum.

235.—Minimum physical properties, as determined on a standard turned test specimen, of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch, 55 000 to 65 000
Elongation: percentage in 2 in..... 28

236.—A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of ½ in., without fracture on the outside of the bent portion. The bending may be effected by pressure or by blows.

237.—The number and location of the test specimens to be taken from a melt, blow, or forging shall depend on their character and importance, and, therefore, must be regulated by individual cases. The test specimen shall be cut, cold, from the forging, or full-sized prolongation of the same, parallel to the axis of the forging and half way between the center and the outside; the specimens shall be longitudinal, *i. e.*, the length of the specimen shall correspond with the direction in which the metal is most drawn out or worked. When forgings have large ends or collars, the test specimens shall be taken from a prolongation of the same diameter or section as that of the forging back of the large end or collar. In the case of hollow shafting, either forged or bored, the specimen shall be taken within the finished section prolonged, half way between the inner and outer surfaces of the wall of the forging.

238.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether or not the steel is within the limits in chemical composition specified in Paragraph 234.

239.—Forgings shall be free from cracks, flaws, seams, or other injurious imperfections, and shall conform to the dimensions shown on the drawings furnished by the purchaser, and shall be made and finished in a workmanlike manner.

240.—All forgings shall be annealed.

Axle Steel.

Qualities of Axle Steel.

241.—Axle steel may be made by the open-hearth or crucible process.

242.—Phosphorus ..... 0.05% maximum.
Sulphur ..... 0.05% maximum.

243.—Minimum physical properties, as determined on a standard turned test specimen, of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch	80 000
Elongation: percentage in 2 in	20

244.—A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of 1½ in., without fracture on the outside of the bent portion. The bending tests may be made by pressure or by blows.

245.—Turnings from the tensile test specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether the melt is within the limits in chemical composition specified in Paragraph 242.

Boiler Plates.

Qualities of Boiler Plate Steel.

246.—The steel used for boilers and fire-boxes shall be made by the open-hearth process.

247.—Phosphorus	0.04% maximum.
Sulphur	0.04% maximum.

248.—The physical properties required shall be as follows:

Tensile strength desired, in pounds per square inch, 60 000.

$$\text{Elongation: minimum percentage in 8 in.} = \frac{1\,500\,000}{\text{Ultimate strength.}}$$

Character of fracture	Silky.
Cold bends, without fracture	180° flat.

249.—The ultimate strength shall come within 4 000 lb. of that desired.

250.—Chemical determinations of the percentage of carbon, phosphorus, sulphur, and manganese, shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the Engineer or his inspector. A check analysis shall be made from the finished material, if called for by the purchaser, in which case an excess of 25% above the required limits will be allowed.

251.—Specimens for tensile and bending tests for plates shall be made by cutting coupons from the finished product, which shall have both faces rolled, and both edges milled to the usual form of the standard test specimen, 1½ in. wide on a gauged length of at least 9 in.; or with both edges parallel.

Nickel Steel for Machine Parts.

Qualities of Nickel Steel.

252.—Nickel steel shall be made by the open-hearth process.

	Plates, shapes, and bars.	Rivets.
253.—Phosphorus shall not exceed	0.04%	0.04%
Sulphur " " "	0.05%	0.04%
Nickel, not less than	3.00%	3.25%

254.—The physical properties required shall be as follows:

	Plates, shapes, bars, and forgings, pounds per square inch. Minimum.	Rivets.
Tensile strength.....	80 000	60 000 to 70 000
Elastic limit.....	50 000	40 000 minimum

Elongation, percentage in 8 in., for plates, shapes, bars, and forgings; and also for rivets $= \frac{1\,600\,000}{\text{Ultimate strength.}} = \text{minimum.}$

Elongation, percentage in 2 in., for forgings = 25.

255.—Specimens cut from forgings (1 in. by $\frac{1}{2}$ in.) shall bend, cold, 180°, around a diameter of 1 in., without fracture on the outside of the bent portion.

256.—Specimens cut from plates, shapes, and bars shall bend, cold, 180°, around a diameter of three times their thickness, without fracture on the outside of the bent portion.

257.—Each rivet rod shall bend 180°, flat, on itself, without fracture on the outside of the bent portion.

258.—Rivet rods shall be tested as rolled.

259.—The fracture of all tension tests shall show a fine silky texture, of a uniform bluish gray or dove color, free from black or brilliant specks, and shall show no sign of crystallization.

260.—All nickel-steel forgings shall be properly annealed.

261.—Annealed eye-bars and similar members, when full-sized pieces are tested, shall comply with the following requirements:

Minimum ultimate tensile strength, in pounds per square inch	75 000
Minimum elastic limit, in pounds per square inch.	45 000
Minimum elongation in 10 ft., including fracture.	12%
The fracture shall be mostly silky, and free from crystals.	
Full-sized pieces shall bend, cold, 180°, around a diameter of twice their thickness, without fracture.	

Tool Steel.

Qualities of Tool Steel.

262.—This steel is generally used for parts which require hardening or oil tempering, such as pivots, friction rollers, ball-bearings, and springs.

263.—Tool steel shall be made by the open-hearth or crucible process.

264.—

Carbon	1.00% minimum.
Phosphorus	0.04% maximum.
Sulphur	0.04% maximum.
Manganese	0.50% maximum.

Phosphor-Bronze.

Qualities of Phosphor-Bronze.

265.—Special phosphor-bronze shall be used for high pressures and slow speed.

266.—The metal shall have a minimum elastic limit in compression of 27 000 lb. per sq. in. The permanent set at 100 000 lb. shall not exceed $\frac{1}{10}$ in.

267.—A test piece shall be cut from a coupon to be moulded and cast on some portion of each casting. Test pieces shall be 1-in. cubes, finished.

268.—Phosphor-bronze composed of the following ingredients and of the following proportions has given satisfactory results:

Copper	79.7 per cent.
Tin	10. " "
Lead	9.5 " "
Phosphorus	0.8 " "

Babbitt Metal.

Qualities of Babbitt Metal.

269.—Babbitt metal composed of the following ingredients and of the following proportions has given satisfactory results and a low coefficient of friction (0.03 to 0.04):

Copper	3.6 per cent.
Tin	89.3 " "
Antimony	7.1 " "

Purpose of the Specifications.

270.—It is the purpose of these specifications to provide a first-class structure. They are intended as an aid in designing and fabrication. The subject of machine design is so great and varied that no single work of this character can cover all points. As a further aid in securing a first-class structure, the following works will be considered authoritative, in the order named:

1. Unwin's "Machine Design," Part I, Ed. 1909.
   Unwin's "Machine Design," Part II, Ed. 1902.
2. "A Manual of Machine Design," etc., by Low and Bevis, 11th Impression.
3. Reuleaux's "Constructor," Translated by Suplee.
4. Kent's "Pocket Book," 8th Ed.

271.—Machine parts shall be designed, if practicable, by the methods of applied mechanics, but such designs shall be viewed in the light of experience. It should be borne in mind that machine design is not based on the precise methods in vogue for statical structures.

DISCUSSION

Mr. Smith.

AUGUSTUS SMITH, M. AM. SOC. C. E. (by letter).—The writer desires to add his acknowledgment of the value of the specifications for movable bridges,* by C. C. Schneider, Past-President, Am. Soc. C. E., and of the discussions contributed by other members of the Society on that subject. There was great need for definite specifications, and Mr. Leffler's valuable contribution is another important gift to the Profession.

The writer is glad to discuss a few of the details defined so unequivocally by the author, taking them up in order, as follows:

Paragraph 55.—The author apparently favors the use of tapered keys, but obviously could not attempt, in the text of a specification, to give any justification for this preference. Without rehearsing the arguments against tapered keys,† the writer would call attention to the fact that they cost more than straight keys and require considerable room on the shaft for each wheel fitted, so that it is not practicable to place a wheel close to a bearing, and, when such keys are used, it is not practicable to place two or more wheels very close together. In some cases tapered keys are necessary. The writer's practice is to use straight keys where possible, and tapered keys where straight ones are not practicable.

Paragraph 92.—Worm gearing is used frequently for transmitting power, where it is desirable to hold the load. If the angle of the thread for the worm were 20° or more, the wheel would be able to turn the worm backward. There are not many data on the limiting angle of thread which will give the maximum efficiency of a worm drive without being steep enough to permit the wheel to drive the worm. The writer has found the angle to be approximately 6°, but much depends on the nature of the thrust bearing holding the worm, and the angle would be less if first-class ball bearings were used.

Paragraph 136.—The author allows for the bending stress in the individual wires of rope bent around a sheave, and points out that this is a function of the diameter of the individual wires and not directly of the diameter of the rope, as it is sometimes apparently thought to be. The same thing has been worked out by the engineers of the Trenton Iron Company.‡ Attention is called to the fact that the minimum diameter of sheaves and drums given in the lists of all manufacturers of wire ropes has been apparently agreed on, like the price list, but it is not altogether consistent with the foregoing theory.

The writer confesses to some obtuseness in his power of conception of the action of cumulative stresses. If the bending stress in the

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 258.

† Summarized in Kent's "Mechanical Engineer's Pocketbook."

‡ The results are given in the handbooks of that company.

Mr. Smith.

wires is really added to the tensile stress produced by the load on the rope, an overloaded rope should always break on a sheave or drum, and not in a straight part.

Paragraph 140.—The author's formula for determining the strength of gear teeth follows the general form advocated by Mr. Lewis, namely, by making the allowable pressure proportionate to the pitch and face multiplied together, instead of the earlier formula given by Tredgold, in which the pitch was squared. The author's formula makes due allowance for the number of teeth and the velocity, and is intended to apply to steel gearing, because, according to Paragraph 38, only cast or forged steel should be used for this purpose. He should add a formula fixing the face in terms of the tooth pressure, to provide for wear. Cast iron works with less friction, and usually with less tendency to cut, than cast steel or forged steel when running together, and the writer is doubtful about allowing much greater tooth pressure per inch of face for steel than for first-class close-grained cast iron. It will be observed that the permissible pressure on the contact surfaces between the teeth of gears given in Paragraph 140 is many times greater than that allowed in sliding contact by Paragraph 150. Unwin mentions the old rule that the pressure between teeth should not exceed 400 lb. per in. of face, in order to obtain good wearing results. The late James Christie,* M. Am. Soc. C. E., in his discussion of Mr. Schneider's paper, gives a formula naming the working pressure of the tooth in pounds per inch of face, in terms of the pitch multiplied by a constant dependent on the nature of the material, and divided by the square root of the velocity. Mr. Christie's formula, it will be observed, gives higher values for steel than for cast iron, in the ratio of 10 to 6.

Paragraph 157.—The writer would suggest adding to this paragraph a sentence to the effect that when brakes which act through the transmission machinery are used, all parts of that machinery so affected shall be designed to withstand the maximum force set up by the application of the brakes, if it should be more than that of the driving motor.

Paragraph 168.—The author limits the maximum piston speed of internal-combustion motors to 350 ft. per min. A comparison of the various speeds given by manufacturers of so-called "heavy-duty" motors would indicate an allowable piston speed of 500 ft. per min., and even higher. The writer's experience with a gasoline motor indicates perfectly satisfactory results at 500 ft. per min. One objection to putting the piston speed too low in motors of this type is the inevitable leaks past the piston rings, and at low speeds there is a marked loss in compression around the valves, so that an internal-

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 318.

ombustion engine does not develop the proportional power at a ow speed that it would at a higher speed. Mr. Smith.

The writer would also recommend adding a provision to this para- raph limiting the ignition, if electric, to the jump-spark method, and ould also advocate specifying the use of a jump-spark apparatus in hich the secondary coil is made up on each spark plug as part of it, o that a low-tension current of 6 or 10 volts at the outside is all hat would have to be taken care of in the wiring. He would recom- end a clause prohibiting the use of any internal combustion motor n which cooling water was passed through a packing or joint designed o withstand the gases of combustion. This provision, in the case f engines in which the cylinder head is made separable from the body f the cylinder, would require the cooling water to be by-passed out- de from the cylinder to the head, or brought to and from the head nd cylinder separately. Preference should be given to an air-cooled otor of proven reliability.

Paragraph 177.—In this paragraph the author names the spare arts to be furnished with an electric motor. The writer thinks that ertain spare parts, such as igniters and crank-pin brasses, would be dvisable in the case of internal-combustion motors.

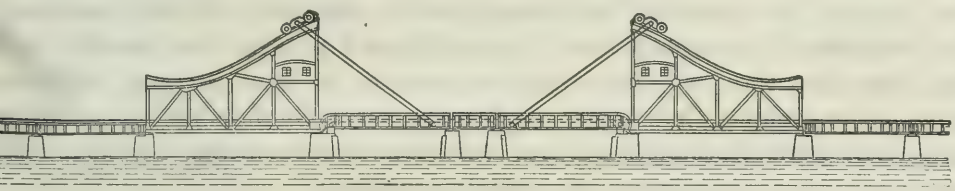


FIG. 3.

Paragraph 180.—The author specifies controllers of the reversible rum type, and while these give very good results, the writer has found hat controllers of the disk type, such as are made by the Electric ontroller and Manufacturing Company, give equally satisfactory re- lts in hoisting and crane service, so that he would not advise limit- g controllers to the drum type.

In the discussion of Mr. Schneider's paper, a number of leading esigns of bascule bridges were brought out. Though it has no direct earing, as part of this discussion, it might be of interest at this time add to the various bridges brought out by Mr. Schneider's paper, design which the writer prepared for the Stone Bridge, at Tiverton, r the State of Rhode Island, based on the old Delille type of fortifica- on draw-bridge.

The mechanical principles involved in this bridge are obvious from ig. 3. The advantage of the stiff connecting link between the mov- g bascule and the counterweight is that the bridge can be balanced all points and forced down against a wind pressure. Another ad- ntage is that the spans on which the counterweight moves can

Mr. Smith. be used frequently as through truss spans, like approach spans, on each side of the draw, with some economy in metal. A third feature is that all the moving parts are high above the water, and cannot be affected by salt spray or ice. In common with many others of the bascule type, a low-level bridge is possible, and no expensive foundations for counterweight or the like are necessary. Another advantage—and it seems to the writer a very important one—is that this bridge, especially where a double-leaf bascule is used, has a more pleasing architectural effect than is commonly the case with bascule bridges.

In the Tiverton Bridge, which was designed for highway and trolley car service, no central pier was used, but the connecting rods were made of very ample cross-section so as to reduce the deflection of the bridge at the center to a negligible amount for the purpose. For railway service, a center pier of some sort would doubtless be desirable; such a pier would be built in the middle of a two-leaf span with less obstruction to traffic than would be the case if a swinging draw were used.

Mr. Mercer. CHARLES H. MERCER, M. AM. SOC. C. E. (by letter).—The writer is connected with a company which fabricates and erects a considerable number of bridges movable in a vertical plane, and, after consulting freely with the electrical and mechanical engineers of this company, has considered it appropriate to compile and present the results, with the hope that they will be found to contain, at least in part, sufficient merit to commend themselves to Mr. Leffler. Such a set of specifications can best become a smooth working document by criticism followed by a broad review of the whole problem.

Paragraph 2.—The "Specifications of the New York Central Lines for Steel Railroad Bridges," for 1910, are made a part of the author's specifications. These specifications require the railroad company to furnish and lay the track, rails, timber floor, etc. This is logical for an ordinary steel bridge, but does not seem to be proper for a counterweighted bridge. The contractor for a bridge of this character cannot balance the bridge until the floor is laid. If the railroad does this work, the labor forces will be mixed, which is objectionable. Besides, the majority of these bridges are erected open, which tends to increase the complications if the railroad company lays the track. The specifications would be more harmonious if they required the contractor to do this work.

Paragraph 21.—The constants given in this paragraph do not seem to be sufficiently specific; thus, for rolling friction, the radius of the roller and whether one or two faces are in contact should be considered. The writer has had to do with "movable" bridges moving on rollers of 12 and 100 in., respectively, and with segmental rollers of much larger diameter. Although there is no agreement on the constant used in the several formulas for rolling friction, it would seem that

Mr. Mercer.

the maximum of these could be used advantageously in ascertaining the resistances to be overcome.

The loss caused by the stiffness of wire cables depends on the sheave diameter, the number and angle of the bends, and the type of rope. The usual limitation of sheave diameter is not made in these specifications, probably because the stresses due to bending of ropes over them are taken into account. It would seem, however, that the sheave diameter should be considered in calculating the power requirements.

Paragraph 22.—The coefficient for journal friction is given as 0.07. The writer thinks this should be covered by Paragraph 21, using the same constant as for friction on trunnions, that is, $\frac{1}{8}$ for starting and $\frac{1}{16}$ for running.

Paragraph 23.—This paragraph reads: "The time to open the bridge after the ends are released shall be as specified on the proposal drawing."

The writer suggests that it be changed to: The time to open the bridge after the ends are released shall be approximately as specified on the drawings.

The writer has had to do with contracts for two bridges under these specifications with the company represented by Mr. Leffler, and there has been no thought of taking exception to this paragraph, nor need there be when the specifications are interpreted broadly by a reputable engineer; but the specification is now apt to be more widely adopted and made a part of contracts. In such a case, could not an unscrupulous representative of a buying company make the contractor suffer if the bridge required somewhat more time to open than was specified? Bascule and lift bridges are not designed by contractors, but ordinarily by the patentees, who are experts for their respective structures, and it is a proper assumption that they are more expert on these special bridges than the contractor's corps of engineers. The specifications and plans on which the contractor bids are complete, specifying in detail the requirements of the prime mover, the size of all pinions, gears, shafting, etc., and the sectional areas of all the component members, as well as the details for their connection. The contractor is told just what to do. Except for workmanship and materials, it does not appear to be equitable that he should be held responsible for results. A contract based on divided responsibility is wrong in principle; let the designer be responsible for the design and the contractor for the construction and materials. The writer feels that, in specifications of this character, the portion covering design should be separate, and the contractor should not be responsible for it if designed by any other than himself.

Paragraph 36.—The writer believes that trunnions bearing on phosphor-bronze should be of axle steel rather than structural steel. The harder steel would be less apt to cut and score.

Mr. Mercer.

Paragraph 126.—This paragraph provides for tests of trunnions on their bearings, and the test requires special apparatus, with consequent delay and expense. The paragraph might well be omitted, but if there is a demand for some such test, the results sought could be attained by placing the trunnions in their respective bearings on a surface plate.

Paragraph 156.—This paragraph states that: "All machinery parts shall be designed with sufficient strength to resist the greatest pressure which can be exerted by the motor." Paragraph 173 specifies an overload for direct-current motors of 200% for 3 min., and 400% momentarily. Apparently, this overload for the motors is simply a test of the insulation, but, taking the requirements of Paragraph 156 literally, the machinery would have to be designed for an overload of 400% with the units specified in Paragraphs 133 and 135, which results would be abnormal.

Paragraph 168.—The maximum piston speed of the gasoline engine is lower than required by standard practice. In all cases the piston speed should depend on the size of the engine. The advisability of operating the locking machinery by the gasoline engine is doubted. If operated by hand, the machinery would be much simplified, and there would be much less likelihood of jamming the lock machinery when undertaking to drive home when not in proper position.

Paragraph 169.—"Water-proof" should be changed to "weather-proof," as it is better to have a semi-open type of motor properly housed. In the case of a totally enclosed motor, the operator seldom inspects it as thoroughly as he would if he could examine the working parts without removing the cover.

Paragraph 172.—The words, "direct-current," in the first line should be omitted.

Paragraph 173.—It would be much clearer if, for each bridge, a definite statement was made as to how much torque the motor had to develop in foot-pounds for 3 min., and the exact time the motor would have to carry the normal torque of 400 per cent. These values should then be selected so that a standard motor which fulfills one will fulfill the other without a special arrangement of the winding.

The percentage of drop in voltage which will be allowed under full-load conditions, between the switch-board and the motor terminals, might be stated for each proposition.

Paragraph 175.—This paragraph is applicable to a back-geared motor. It may not be advisable to use such a motor in every case.

Paragraphs 181 and 182.—These paragraphs are slightly confusing. Paragraph 181 calls for a controller for each motor. Paragraph 182 calls for series-parallel control, which can be applied to two or more motors. It might be better to have the control specified for each case, because, where the operating machinery does not rotate with the bridge, it would seem as if a remotely controlled contact or board

ould be used to advantage, especially if the operator's house is some istance from the bridge. Mr. Mercer.

Paragraph 188.—The writer believes the motor-operated, emergency rake to be too complicated for the average operator. A better brake, s far as concerns reliability, less complication, and being more nearly alanced in all positions of the bridge-leaf, is obtained with the same ype as that used for the operating motors, provided the size required oes not exceed the largest size manufactured. In case it does, two uch brakes should be used.

Paragraph 190.—It seems to be desirable that the emergency-brake witch be in closed position, thus releasing the brake before power s available for the operating motors. In this case the switch would e electrically interlocked, but the operator would be free to apply he emergency brake at any time, regardless of the position of the ontroller arm.

Paragraph 192.—Submarine cables, if needed, should be furnished nd laid by the contractor for almost the same reasons as those given or laying the floor, and to obviate division of responsibility due to ossible delay. The trench for such cables if required, might be redged or dug by the railroad company.

Paragraph 193.—For bridges moving in a vertical plane, the logical lace at which to deliver the current to the electrical contractor is t the switch-board.

Paragraph 197.—If the electrical equipment is installed as re- uired in these specifications, there seems to be no need of requiring verything to be in accordance with N. E. C. Standard, as this standard s devised for installations where the danger of fire is great, which is ot true on a bridge. Bridges were not considered in making up he N. E. C. Standard, and its application to them is not sufficiently efinite.

Paragraph 212.—In measuring currents, it seems to be standard ractice to use shunts for the ammeter, thus avoiding heavy currents hrough the measuring instrument and the consequent impairment hereto.

Paragraph 217.—The type of the alternating-current motor will epend on the kind of current available, and need not be a 25-cycle, 20-volt, three-phase type.

Paragraphs 265 and 266.—Phosphor-Bronze.—It is well to preface ny discussion on this subject with a statement of the fact that it s most difficult to obtain consistent physical results from any given hosphor-bronze alloy, and the most expert superintendence in the oundry is required to produce uniform results.

Probably on account of the comparatively small quantity of hosphor-bronze used in movable bridges, it has not received the at- ention from bridge engineers that its importance warrants. In look-

Mr. Mercer.

ing over bridge specifications in which the composition for phosphor-bronze is given, the writer finds that almost all specify that given in Paragraph 268 of the author's specifications, namely:

Copper	79.7	per cent.
Tin	10.0	" "
Lead	9.5	" "
Phosphorus	0.8	" "

and this composition will be referred to hereinafter as "standard."

Practically all the bearings in movable bridges may be characterized as high-pressure and slow-speed, as against low-pressure and high-speed. For the latter conditions, the "standard" composition seems satisfactory, and specimen tests will give physical results practically as follows:

Compression:

Elastic limit, in pounds per square inch	13 000 to 15 000
Permanent set from 100 000-lb. load, in inches	0.27

This elastic limit is based on a set of 0.001 in. in 1 in.

Tensile:

Yield point, in pounds per square inch	20 000
Ultimate, in pounds per square inch	25 000
Elongation, percentage in 2 in	4
Reduction	3 per cent.

After discussing this subject thoroughly with a leading metallurgist, the writer submits the following composition for high pressure and slow speed:

Copper	85.2	per cent.
Tin	14.0	" "
Phosphorus	0.8	" "

The approximate physical results from this composition on a cylinder with an area of 1 sq. in. and 1 in. long, should be:

Compression:

Elastic limit, in pounds per square inch	20 000 to 22 000
Permanent set from 100 000-lb. load, in inches	0.12 to 0.16

In tension tests, this composition will show some ductility.

The presence of lead in bridge bearings subjected to heavy pressures is objectionable, for the following reasons: Lead does not alloy with the copper-tin bronzes, but is mechanically admixed, and, being a very soft metal, it immediately lowers the elastic limit in compression, causing the bronze to become plastic and flow under pressure. It likewise, being a soft metal, tends to grip or seize the shaft under conditions of heavy pressure and poor lubrication. The harder the

bronze the less likely it will be to seize the steel shaft, providing, of course, the bronze does not exceed the steel in hardness. Mr. Mercer.

Experiments made by the late C. B. Dudley,* M. Am. Soc. C. E., indicate that the relative wear of a copper-tin alloy is one and one-half times that of the "standard" copper-tin-lead alloy.

For convenience, Table XCIII, showing the results of these experiments, is here quoted.

"TABLE XCIII.—Relative Wear of Bronze Bearings (*Dudley*).

Alloy tested.	Composition.					Relative wear.
	Copper.	Tin.	Lead.	Phosphorus.	Arsenic.	
Standard lead (phosphor) bronze S	79.70	10.00	9.50	0.80	....	1.00
Ordinary bronze	87.50	12.50	....	....	....	1.49
Arsenic bronze, A	89.20	10.00	....	....	0.80	1.42
Arsenic bronze, B	82.20	10.00	7.00	....	0.80	1.15
Arsenic bronze, C	79.70	10.00	9.50	....	0.80	1.01
Bronze K	77.00	10.50	12.50	....	....	0.92
Bronze B	77.00	8.00	15.00	....	....	0.86

This table was determined by placing a "standard" and an experimental bearing on opposite ends of the same axle on locomotive tenders. From time to time they were removed and weighed and the wear was recorded. The copper-tin alloy also gave some trouble from heating. These experiments were conducted about 1892.

These results as to relative wear were confirmed by Clamer about 1903, and are set forth on page 426 of the book:

"He used a specially designed friction-testing machine, with test-bearings, measuring $3\frac{1}{2}$ inches x $\frac{1}{2}$ inch. In each experiment the journal ($3\frac{3}{4}$ inches in diameter) made 100 000 revolutions at a speed of 525 revolutions per minute, the same pressure (1 000 lbs. per square inch), oil, and method of lubrication being used throughout. It was found that the rate of wear diminished, though the friction and temperature increased, as the percentage of tin in the bronze was decreased and the percentage of lead increased (see Table XCIV)."

"TABLE XCIV.—Relative Friction and Wear of Lead-Bronze Bearings (*Clamer*).

	Copper.	Tin.	Lead.	Friction, lbs.	Temperature above room, °F.	Wear, in grms.
1	85.76	14.90	.....	13	50	0.2800
2	90.67	9.45	.....	13	51	0.1768
3	95.01	4.95	.....	16	52	0.0776
4	90.82	4.62	4.82	14	53	0.0542
5	85.12	4.64	10.64	$18\frac{1}{2}$	56	0.0380
6	81.27	5.17	14.14	$18\frac{1}{2}$	58	0.0327
7	75. ?	5. ?	20. ?	$18\frac{1}{2}$	58	0.0277
8	68.71	5.24	26.67	18	58	0.0204
9	64.34	4.70	31.22	18	64	0.0130

* "Lubrication and Lubricants," by Leonard Archbutt and R. Montford Deeley, p. 425.

Mr. Mercer.

It is to be particularly noted from this table (XCIV) that the friction on copper-tin-lead alloys is approximately 50% greater than on copper-tin alloys, which approximate the writer's composition for high-pressure and slow-speed bearings.

The results, as set forth in the Clamer's Table XCIV, were confirmed later by Spare, from experiments made on the same friction testing machine, namely, the Cornell machine.

The conditions under which the experiments were made by Dr. Dudley, and Messrs. Clamer and Spare differed widely from those which exist in a bearing with a load of, say, from 1 600 to 3 000 lb. per sq. in. and supporting a trunnion which revolves only through 80° during the operation of the bridge. The amount of wear on the copper-tin alloy bearing with 100 000 revolutions, as reported by Clamer, was not appreciable from a practical standpoint. With an average of ten openings per day, for 250 days per year, it would take 40 years for 100 000 operations of a bridge. Therefore, the problem for high pressure and slow speed is entirely different from that covered by the experiments. It has not been overlooked that the frictional area for a certain number of revolutions varies with the size of the journal or trunnion.

It seems obvious that, for high-pressure and slow-speed bearings, stiffness combined with a low frictional resistance is essential, and the wear is unimportant. Stiffness is shown in the composition submitted by a high elastic limit in compression, while sufficient ductility is retained.

To summarize: the writer recommends for movable bridge bearings a copper-tin alloy with a maximum content of 1% of phosphorus (other alloys in excess of one-half of 1% will not be permitted), with physical results specified as follows:

Compression:

Elastic limit, in pounds per square inch	19 000 to 23 000
Permanent set from 100 000-lb. load, in inches	0.12 to 0.16

Tension:

The yield point, ultimate, and elongation in 2 in. to be recorded.

The compression test should be made on a cylinder having an area of 1 sq. in. and 1 in. long and the tension test on a specimen having a diameter of ½ in. The compression elastic limit should be determined by the load that gives a permanent set of 0.001 in. It may be noted that the physical tests proposed by the writer are nearly a mean between the physical requirements of Paragraph 266 and the physical

results which may be expected from the "standard" composition given in Paragraph 268. Mr. Mercer.

The writer acknowledges his indebtedness to Mr. C. R. Spare, Holmesburg, Philadelphia, Pa., for information on the subject of phosphor-bronze.

P. J. Reich, M. Am. Soc. C. E. (by letter).—Mr. Leffler's "Specifications for Bridges Movable in a Vertical Plane" are quite complete. In the following the writer suggests a few insertions which would eliminate possible misunderstandings; questions the advisability of a number of unusual limitations; and brings up one general question—the greatest power to be assumed as being exerted by the motors, in the calculations of the machinery to hold it, a point on which there seems to be no uniformity. Mr. Reich.

Manner of Bidding.

The classification of the materials in detail for payment is very good, and is to be recommended in preference to the more general classifications that are frequently used. It would be advisable in Paragraph 3 to exclude from the machinery—in addition to the electric motors—all gas and steam engines, together with their equipments. These are usually purchased by the bridge contractor, ready for operation. As the weights are not ordinarily at hand for estimating, it would be preferable and more convenient to place a lump-sum price on each, with its equipment.

Paragraphs 6 and 7.—It would generally be preferable to omit "sockets" from Paragraph 7, and include them in Paragraph 6, with wire ropes, as they are furnished and attached to the rope by the wire rope sub-contractor.

Paragraph 8.—It is probably the intention of Paragraph 8, taken in connection with Paragraph 3, that attached machined parts, such as cast-steel sleeves, bronze bushings, etc., for movable joints in links and similar members, be weighed separately and classified as machinery. A note added to this paragraph would prevent any misunderstandings.

Similarly, in Paragraph 10, it is advisable to add that "treadplates" are to be included with the segmental and horizontal girders.

Several minor items not mentioned for payment are: Operator's house with its fixtures, machinery housings, and lumber, each of which should be paid for at separate lump-sum prices. (Structural steel in operator's house, or machinery housings should be classed with structural steel.)

Gas-pipe railings, which are usually expensive, should be paid for at a separate price per pound, the size of railing being specified.

Mr. Reich.

Operating Machinery.

Paragraph 36.—"Trunnions, pins, and shafting more than 3½ in. in diameter shall be of forged structural steel." It is questionable whether enough good comes from this low limitation to warrant the additional expense and delays incident to getting them. The writer has used the rolled rounds up to 5 and 5½ in. with entire satisfaction.

Paragraph 39.—In limiting pinions to forged steel, it would be necessary to make exceptions for those of unusual size, allowing them to be cast. Exceptions should also be made to the cutting; and cast teeth should be required when the pinions mesh with cast teeth, as would be the case where pinions mesh with the racks.

Paragraph 48.—"Nuts shall be secured by split pins put through the bolt." This may be interpreted in two ways: As a general note for all bolts; or only for those where nuts are apt to work off. To make it a general requirement would be an unnecessary expense, and it is suggested that they be used in special cases only where requested by the engineer.

Paragraphs 55 to 59.—Proportions, etc., of Keys.—As many of the bridge companies have standards for keys, to which they are accustomed, and which approximate very closely those outlined, it would be preferable to make the specifications a little more flexible, permitting their use when acceptable to the engineer. This should not be objectionable, as the theoretical requirements are taken care of under Paragraph 56. Gib heads on keys are not always reliable, as they frequently break off. Further, gib-head keys are not as simple to make and fit as those without. Much better results would be obtained by requiring keys to be placed so that they can be backed out, using the gib-head keys only where this cannot be done.

Paragraph 62.—"Keys shall be held in place by set-screws." This seems to be an unnecessary general requirement, in view of the fact that the machinery ought to be in charge of a competent man who should go over it frequently enough to detect any key which might be working out. It is doubtful if a key that would work loose would be held long by a set-screw. Where keys might readily drop out, should they become loose, as in the case of a vertical shaft, effective means should be used for holding them.

Paragraph 68.—This paragraph limits the use of Babbitt metal to shafts 3 in. or less in diameter. When the working pressures do not exceed 500 lb. per sq. in., Babbitted bearings are entirely satisfactory for the larger ones, are easy running, economical, and simple to replace.

Paragraph 77.—"Dust covers shall be provided for principal bearings, in particular for trunnions" (where shown on general plans). It is very desirable, from a contractor's point of view, to have

hese at least indicated where required, and it is suggested to modify his paragraph by adding the words: "Where shown on general plans." his is also referred to when discussing Paragraph 93a. Mr. Reich.

Paragraph 86.—Taking care of the effect of key-ways, could also be xcepted for overhanging shafts, in which case the maximum moment at the center of the bearing where no key-ways are cut.

Paragraph 88.—This paragraph limits gear teeth to 20° involute. he differences and relative advantages of the 15 and 20° involute eth over one another are not sufficient to exclude one or the other. n the past, the 15° involute was the more commonly used, and hence ore bridge shops are equipped to cut them. A change means a large xpense, which is out of proportion to the advantages gained.

Paragraph 92.—This paragraph limits the angle of thread of worm earing to 20° as a minimum. As far as wear and efficiency are con-rned, an angle of thread of from 13 to 15° gives very satisfactory sults, and ought to be permitted. The use to which the worm gearing put also governs, to some extent. If the span were coasting, a wheel the main train of gearing would of necessity turn the worm. The riter has no data as to just what this angle would be.

Paragraph 92a.—The writer has frequently used 24-toothed worm heels, and knows of some 21-toothed wheels in service, working satis-actorily. It would seem that 28 as a minimum might be high.

Paragraph 93a.—The contractor should be asked to furnish only ich safety guards as are shown on the general plans. To include l, from a general clause such as this, may mean much or little, epending on the engineer in charge. The addition of these, or changes their location after the detailed plans are completed, almost in-riably costs the contractor more, in his drafting department, than would to take care of it entirely at the bridge site after the erec-on of the work. It would be far more satisfactory to the contractor the safety guards referred to in Paragraph 93a, the dust covers Paragraph 77, and railings in general which are not shown on the ans, were taken care of entirely by the purchaser. In the end, would be the more economical to the purchaser, would ordinarily oid delays, by removing uncertainties, and they would unquestionably more suitably placed.

Workmanship.

Paragraph 121.—Although it is not definitely permitted, the writer ould infer from this paragraph that racks with their pinions are have cast teeth.

Motors.

Paragraph 156.—"* * * the motor shall be of ample capacity move or turn the bridge at the required speed. All machinery parts all be designed with sufficient strength to resist the greatest pressure

Mr. Reich. which can be exerted by the motor." It is very desirable that a method of obtaining the horse-power of the motors from the torque curves of Paragraph 31, be fixed by the specifications; and, further, if any additional allowance of power is to be made for a margin of safety it should be stated. The fixing of this is essential to another important requirement: The designing of the machinery to enable it to resist the greatest pressure which can be exerted by the motor.

The assumptions to be made for calculations to take care of holding the motor are many, and it would be advisable to have it fixed by the specifications. It is very desirable to have it definitely fixed how the over-loads which the motors must stand should be taken care of in the machinery, giving permissible increases in the unit stresses, etc. These over-loads for which the motors are good are large, as given in Paragraph 173.

The design of the machinery is largely dependent on the assumptions made for these two items, they being somewhat dependent on each other. It is very desirable, therefore, that they be given explicitly in the specifications.

A common method of taking care of the two items is to make the horse-power of the motors sufficiently large, using the maximum torque, as given by Paragraph 31, and the maximum speed as a basis for getting the horse-power. All machinery is proportioned for the normal torque of the motor, and, as low unit stresses are used for the machinery, there is no doubt that the machinery will hold the motor. There may be times, if electrically operated, that the higher starting torque may increase the unit stresses used, but this can be regulated by having the automatic circuit breaker arranged so that the time for opening or closing will approximate the calculated time.

More accurate methods may be devised, depending on the actual torque curves of the motors, but this is objectionable, as the motors are usually the last thing ordered and wanted; and, further, it would delay the progress of the design too much to get these at the start.

Paragraphs 166 to 168.—The last sentence of each of these paragraphs requires the contractor to furnish a work-bench with a full set of machinist's tools, etc. It would be preferable to have these furnished by the purchaser, as the exact needs could then be taken care of, this being dependent, to some extent, on the operator and the proximity of a good machine shop. It is objectionable to the contractor to furnish these, as the final cost is an uncertainty, because of uncertain requirements; further, when they are furnished with the bridge, they are usually carried away by some one before bridge is taken over.

Paragraphs 197 and 216.—Any special requirements by city codes for the electric installation, if the bridge is subject to city authority, or any Government or other harbor special requirement for channel lights and signals, should preferably be furnished by the purchaser a

e time of asking bids. The purchaser is usually in a better position Mr. Reich.
get this information, and it means a saving of considerable trouble
each of the bidders.

Paragraphs 160 to 220.—It would be very desirable, from a con-
ictor's point of view, if Paragraphs 160 to 220, covering power and
erator's house equipment, were divided into three distinct and sepa-
e clauses: One for steam-engine equipment, one for internal-com-
stion motor equipment, and one for electrical equipment. The
ecifications are nearly in this order at present. The separation is
sired for simplicity and to make it unnecessary to pick out the parts
at might be interpreted as being required.

These specifications cover a first-class bridge which would be used
a main line of a railroad over a stream where navigation requires
equent opening. A supplement might profitably be added, waiving
rtain restrictions in the design and workmanship of the ma-
inery and equipment, for economical purposes, for bridges not on a
in line, or for those which are not opened frequently.

In conclusion, the writer would add that one essential feature to the
ccess of movable bridges, to which not enough attention is ordinarily
ven, is the erection. Many of the troubles ordinarily credited to
ner sources would be eliminated entirely if a good competent ma-
inist, familiar with this class of work, were added to the regular
rces, to superintend the installation of the machinery.

B. R. Leffler, M. Am. Soc. C. E. (by letter).—The writer desires Mr. Leffler.
express his appreciation of the discussion of his paper, and offers
e following notes on some of the points brought out.

As a result of the discussion and the writer's experiences since the
per was written, the specifications have been revised and are pre-
ted in separate form.

Taking up Mr. Smith's discussion:

Paragraph 55.—The writer sees no objection to other forms of keys,
vided they are of the right proportions. However, a tapered key
h a moderate slope, and held by a set-screw through the hub, is
te adequate. The specifications have been modified to allow the
of other keys.

Paragraph 92.—The writer believes that the only justification for
ng a worm gear, in the bridges covered by the specifications, is for
purpose of getting a large reduction in a limited space where a
in of gears could not be used. For this case, efficiency is of more
portance than the holding power of the worm. In swing bridges the
rm has been used to raise the ends of the bridge; here the holding
ver is of much importance.

Paragraph 136.—The writer does not believe it advisable to look
the handbooks of wire rope makers for the proper diameter of

Mr. Leffler. sheaves for counterweight cables. It seems that a rational theory which gives safe design is preferable to tabulated empirical data.

The writer's knowledge concerning wire rope is not extensive enough to say just where a rope will break. Apparently, it should break where it starts to bend over the sheave. Over the sheave, the frictional resistances of the rope on the sheave groove probably strengthens the rope. Over the sheave, the rope is subject to radial forces which modify the conditions.

Paragraph 140.—The character of the stresses in gear teeth at their contact is unknown. Line contact with sliding is found; the intensity of the stresses between any two bodies pressed together on a line is not known. This is the problem of designing a roller for heavy loads. The best pair of meshing gears, according to the writer's experience, is that of a wooden-toothed bull-wheel and cast-iron pinion.

Teeth designed by the Lewis formula seem to be satisfactory. In the writer's experience, teeth seem to break instead of wearing out. Recently, in the horizontal rack of a rolling lift bridge under the writer's care, several teeth were broken on account of the sudden application of the brakes; the material was cast steel.

For other points by Mr. Smith, the specifications have been modified.

Referring to Mr. Mercer's discussion, the following views are presented:

Paragraph 21.—In rolling bascule bridges, pure rolling is not found. For the full roller, a flange is used; for the segmental roller, teeth in the track. Hence, there is some sliding friction. Moreover, there is considerable deformation, which becomes partly permanent. In one case under the observation of the writer, the rolling structure refused to close without the application of extra torque; this is due to permanent deformation at the end of the track. Some rolling structures are not precise in their movements; the free end is forced sidewise into its closed position by guides. In lieu of more exact knowledge, the coefficient of $\frac{1}{12}$ is a workable one. A coefficient for pure rolling is added.

The coefficient for stiffness of wire rope is not well known. The efficiency of a wire rope belt over two pulleys may be equal to 96% including journal friction. *Eytelwein's formula for stiffness is $R = \frac{Kd^2}{D} P$, in which K is a constant; d is the diameter of the rope, and D is that of the sheaves, both in inches. P is the weight lifted. The pull, which lifts the weight, must equal $R + P$. K can be taken at $\frac{1}{3}$. The specifications are revised accordingly.

* References: Church's "Mechanics of Engineering," 1890, p. 193.
Kennedy's "Mechanics of Machinery," 4th Ed., Art. 80.

Mr. Leffler.

The ratio of the diameter of the sheave to that of the rope in the Halsted Street lift at Chicago is 90. After 18 years of service, the ropes were still in use. A high ratio should be used to limit the friction, and consequent wear, between the wires. A paragraph has been added to the specifications to cover this. According to Rankine, for ratios above 140, the effect of rigidity is negligible.

Paragraph 22.—Journals are not subject to the high pressure found in trunnions; the coefficient at starting is not as great. Again, there is some play in a train of gears, and the motor overcomes the initial journal friction before the initial trunnion friction comes into play.

Paragraph 23.—It is desirable to define clearly the line between the designer and the fabricator. The writer accepts a revision. However, the writer has found it necessary to make specifications for the designers.

Paragraph 36.—The writer recommends the use of a softer phosphor-bronze instead of a harder steel.

Paragraph 168.—The writer has one swing bridge operated by a gasoline engine. The rail-locks are of the sliding-sleeve type, and are actuated by the engine. The gasoline engine is not a suitable type of prime mover for a bascule bridge; it does very well for a lift, if placed on the moving span, and in this position it can be made to operate end-locks for rails and ends of span.

For bridges of the type herein discussed, sliding rail-locks may be dispensed with. The writer has devised a lock which holds securely the ends of the rails, and bridges the gap between the shore rail and the bridge rail, and no machinery is necessary for its operation.

Paragraph 192.—The writer is not prepared to give a specification for submarine cables. It is the practice of the railroad company with which he is connected to furnish the cable, because such cable contains the wires for signal, telegraph, light, and telephone services, as well as power wires for the bridge.

Paragraph 197.—The writer believes that the National Electric Code represents good practice. Every bridge has an operator's house, or power station, in which part of the electric equipment is installed. Obviously, the installation in the house should be first class, from a fire-protection standpoint.

There are many excellent provisions in the Code which should be used in any first-class electric installation. The character and installation of circuit breakers, fuses, switches, motors, etc., are important. To cover all these points in a specification would be a useless repetition of what is already recognized as first-class practice. Hence, a mere reference to the Code should be sufficient.

Paragraph 266.—The writer believes that the suggestions for phosphor-bronze are excellent. The physical qualities of phosphor-bronze suitable for heavy bearings seem to be somewhat unknown.

Mr. Leffler.

The writer has under observation a trunnion which became abraded. The phosphor-bronze showed a permanent set of 0.06 in. on a 1-in. cube under a 100 000-lb. load. Another, and similar, case has occurred. Hence, it seems that a somewhat softer phosphor-bronze is more suitable. The specification suggested is adopted.

The writer has not commented on all the points raised by Mr. Mercer. Nearly all those omitted have been adopted in the revised specification.

Referring to Mr. Reich's discussion, the suggestions are excellent, and the writer has adopted most of them in the revision.

Paragraph 68.—Babbitt metal gives good results if properly poured, but where accuracy of alignment and correct relative position of the machinery parts are to be maintained, bronze and brass linings are better.

Paragraph 88.—The proper angle of obliquity in involute teeth is still unsettled, although the matter has been before the American Society of Mechanical Engineers for several years. For an angle less than 20°, it is necessary to modify the outline, to avoid interference. Even for 20°, a slight change must be made for a pinion of 17 teeth or less meshing with a rack.

The writer finds that the special gear-cutting concerns readily undertake to cut gears of any obliquity without much extra expense. *The present trend is toward an angle of 20° as a standard.

Paragraph 92.—For worm gears, used in bridges of the character specified, efficiency is of more importance than holding power. An angle of thread of 20° is about the lowest limit for a loss of 30 per cent.

Paragraph 156.—The writer has attempted in the revision to clear up the question of designing for excess torques.

The use of the term "Horse-power" is somewhat misleading. It is better to specify the capacity of a motor by naming the variable torque it should exert for a given time, say twice the normal variable torque shown on the torque curves for a cycle of operation.

The writer believes that the torque curves should be worked up as quickly as possible. This is necessary for the correct determination of the gear ratio, and the capacity of the storage battery, if this is intended as the source of power.

Regarding a specification for bridges of minor importance; it is the writer's experience that such bridges give the most trouble in maintenance. Any power-operated bridge should be opened at least once a week, in order to keep the parts in smooth running condition. A movable bridge, to fulfill its purpose, must be in first-class condition continually, even if seldom moved. After a number of years of experience in maintaining movable bridges, it is the writer's opinion that it is impossible to keep a power-operated bridge too efficient.

* In the "American Machinist Gear Book," pp. 20 to 26. the subject is fully discussed.

Mr. Leffler.

The writer begs to submit a demonstration of the formula for the bending of wires, given in Paragraph 38. This is largely of academic interest. The Trenton Iron Company, in its "Wire Rope Book," gives a formula which is intended to serve the same purpose.

In attempting to reduce the Trenton formula, it was found to be erroneous. *Further investigation led to the derivation of the formula given in Paragraph 38. In the deduction a slightly different limiting value of the angle is adopted. This gives a closer approximation.

The Deduction of the Value of R in Paragraph 138.—

Proposition.—A wire is bent over a sheave through a small angle. Determine the least radius of curvature, R, of the bend.

Let Fig. 4 represent the conditions, considerably distorted for purposes of reasoning.

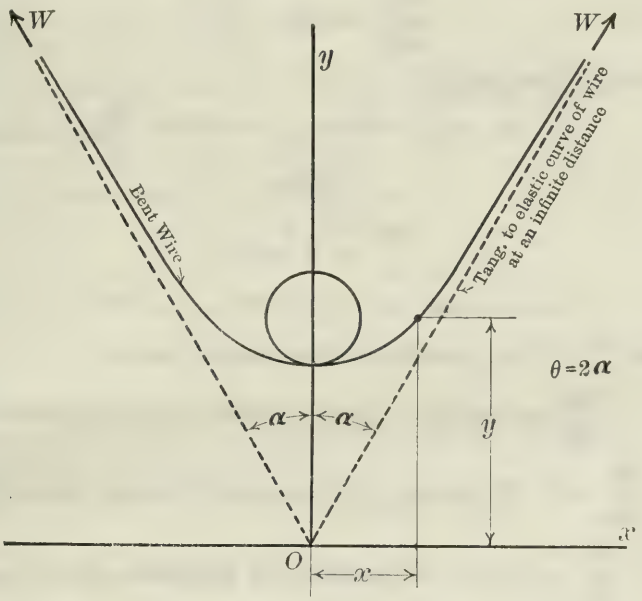


FIG. 4.

For practical purposes, θ is the angle between the straight parts of the wire.

Let I = Moment of inertia of the wire.
M = Bending moment in the wire at any point.
E = Modulus of elasticity.
W = Pull on wire.
S = Length of curve, measured from the y axis.
$RM = EI$.

Now, by calculus,

$$R = \left(\frac{ds}{dx}\right)^3 \cdot \frac{dx^2}{d^2y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$M = W\,(y - x \text{ cot. } \alpha) \text{ sin. } \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

* Private correspondence, dated May 21st, 1912, with Mr. William Hewitt.

Mr. Leffler.

Now, as α is to be a large angle, it will be assumed to be not less than 65 degrees. In Equation (1) $\frac{ds}{dx}$ can never exceed cosec. α in value, or be less than 1: its cube will be practically a constant, say b.

Then $$R = b\,\frac{dx^2}{d^2y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

hence, $$b\,\frac{dx^2}{d^2y}\,W\,(y \sin.\ \alpha - x \cos.\ \alpha) = EI.$$

$$\frac{d^2y}{dx^2} = \frac{b\,W\,(y \sin.\ \alpha - x \cos.\ \alpha)}{EI}.$$

Put $\frac{b\,W \sin.\ \alpha}{EI} = a$, and $\frac{b\,W \cos.\ \alpha}{EI} = h$, then

$$\frac{d^2y}{dx^2} = ay - hx \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Integrating Equation (3),

$$y = c_1\,e^{x\sqrt{a}} + c_2\,e^{-x\sqrt{a}} + \frac{hx}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$e = 2.718 +$$

Now, $$\frac{hx}{a} = x \cot.\ \alpha,$$

hence $$y = x \cot.\ \alpha + c_1 e^{x\sqrt{a}} + c_2 e^{-x\sqrt{a}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Equation (7) is the equation of the elastic curve.

$$\frac{dy}{dx} = \cot.\ \alpha + c_1\sqrt{a}\ e^{x\sqrt{a}} - c_2\sqrt{a}\,e^{-x\sqrt{a}} \quad \ldots\ldots\ldots\ldots (8)$$

For $x = 0$, $\frac{dy}{dx} = 0$, Equation (8) becomes $0 = \cot.\ \alpha + c_1\sqrt{a} - c_2\sqrt{a}$.

For $x = \infty$, $\frac{dy}{dx} = \cot.\ \alpha$, Equation (8) becomes $\cot.\ \alpha = \cot.\ \alpha + c_1 \infty$.

Hence, $c_1 = 0$, and $c_2 = \frac{\cot.\ \alpha}{\sqrt{a}}$.

Equation (7) now becomes

$$y = x \cot.\ \alpha + \frac{\cot.\ \alpha\, e^{-x\sqrt{a}}}{\sqrt{a}}.$$

For $x = 0$, $y = \frac{\cot.\ \alpha}{\sqrt{a}}$, this is the value of y where the curve crosses the y axis.

Equation (1a) now becomes, for R at the y axis, by putting $x = 0$, and $y = \frac{\cot.\ \alpha}{\sqrt{a}}$ in Equation (3),

$$R = \frac{b}{\cot.\ \alpha\ \sqrt{a}} = \frac{\sqrt{b}}{\cos.\ \alpha}\sqrt{\frac{EI \sin.\ \alpha}{W}}.$$

This is the least value of the radius of curvature. Mr. Leffler.

As the limiting values of $\frac{ds}{dx}$ are 1 and cosec. $\alpha = 1.10$, we will

sume as an average value for $\frac{ds}{dx}$, 1.05, hence, as $\left(\frac{ds}{dx}\right)^3$ was called

$\sqrt{b} = 1.08$.

Again, as α is never less than 65°, the average value of its sine can taken at 0.953.

$$\text{Then } R = \frac{1.08 \times 0.976}{\cos.\ \alpha}\sqrt{\frac{EI}{W}} = \frac{1.054}{\cos.\ \alpha}\sqrt{\frac{EI}{W}}.$$

ain, $$I = \frac{\pi\, d^4}{64},$$

ıce $$R = \frac{1.054\ \sqrt{\pi}\ d^2}{8\ \cos.\ \alpha}\sqrt{\frac{E}{W}} = \frac{d^2}{4.27\ \cos.\ \alpha}\sqrt{\frac{E}{W}}.$$

It is close enough to take

$$R = \frac{4\ d^2}{17\ \cos.\ \frac{\theta}{2}}\sqrt{\frac{E}{W}}.$$

REVISED SPECIFICATIONS FOR RAILROAD BRIDGES MOVABLE IN A VERTICAL PLANE.

Scope. 1.—These specifications are intended to cover bascule bridges, which are such as rotate about a horizontal axis; and vertical lifts, which are those in which successive positions are parallel.

Deck. 2.—The Contractor shall place and fasten permanently all ties, rails, guards, and other deck material. Usually, the Railroad Company will furnish all the deck material except rail-locks, rails which must be fabricated to fit locks, and special devices to hold the deck in place.

Responsibility. 3.—If complete general proposal plans are furnished to the Contractor, he shall be responsible for only the material and character of the workmanship and installation. However, for any parts of the design not covered by the proposal plan, and for which the Contractor is to make a design, he shall be wholly responsible.

Electrical Equipment. 4.—Unless otherwise specified, the electrical equipment shall include all electrical parts up to, and including, the switch-board. Electrical equipment carrying the current to the switch-board from the source of power will be covered by separate contract.

5.—The specifications of the New York Central Lines for Steel Railroad Bridges, for 1910, shall apply to movable bridges, except as noted herein.

Manner of Bidding.

Parts Classified as Machinery. 6.—Drums, cylinders, eccentrics, trunnions, and their cast supports, shafting, pistons, gear wheels, racks, boxings, bearings, coupling disks, cast sheaves and wheels, worm gearing, valves, pins about the axis of which the connecting members rotate, whistles, ram screws, end bridge-locks, rail-locks, indicators, cranks, axles, hooks, wrenches, and similar parts of machinery which require machine-shop work shall be classified as machinery and be paid for at a common price per pound. Electric motors, or other prime movers, are not classified as machinery. (11).

Sheaves. 7.—The large sheaves of vertical lift bridges the webs and diaphragms of which are built up with plates, angles, and rivets, shall be paid for at a separate price per pound of finished weight, including casings and fastenings to trunnions.

Air Compressor Boilers. 8.—Air compressor tanks and steam boilers shall be paid for at a separate price.

Wire Ropes and Cables. 9.—Wire ropes and cables and their sockets shall be paid for at a separate price per pound.

Pins, Levers, etc. 10.—The pins, equalizing levers, and cable attachments to the trusses and counterweights shall be paid for at a separate price per pound.

Structural Steel Parts. 11.—Structural steel supporting the machinery proper, counterweight frames, counterweight trusses, steel in operator's house, tower

links, shall be classified as structural steel and be paid for at same price per pound as for the span itself. Attached ma- parts, such as sleeves, bushings, etc., shall be classified as inery. (6).

Operator's House.

.—The operator's house, except for the structural steel therein, be paid for on a lump-sum basis.

.—Structural steel which can be fabricated by the common methods, as punching, reaming, drilling, shearing, planing, etc., usually done for stationary structures, shall be classified as ural steel and be paid for at the same price per pound as for the itself.

Segmental Girders.

.—Segmental girders in rolling bascule bridges and the hori- l girders on which they roll shall be paid for at a separate price ound. This does not include any bracing, floor system, or other ural members which may be attached. Track plates shall be ded.

Hand-Rail.

.—Hand-rail shall be paid for at a separate price per pound.

Electrical Equipment.

.—Electrical equipment, such as wiring, switch-boards, control- lights, blow-outs, cut-offs, solenoids, switches, motors, etc., shall id for on a lump-sum basis.

Engines.

.—Internal-combustion engines and steam engines shall be paid n a lump-sum basis.

.—Cast-iron parts used in counterweights shall be paid for at a ate price per pound.

.—Concrete in counterweights shall be paid for at a price per yard in place.

Extra Parts, etc.

.—It is to be understood that if any extra parts are needed, or uestion arises, all difficulties shall be settled on the pound price as quoted and accepted for the parts in question.

General Details in Designing.

Self-Centering Devices.

.—Self-centering and seating devices shall be used on the free of the moving span. Holding and forcing-down devices shall be for the free ends of each truss.

Rail-Locks.

.—Designs for bridging the gap between the shore rails and ng rails shall be furnished by the Railroad Company. Loose rails not be allowed.

Air Buffers.

.—Air buffers shall be furnished at the free ends of the moving

Counter-weights.

.—The counterweights shall be easily adjustable. Usually, this be done by adding or taking away cast-iron parts, or small con- blocks.

Stairways.

.—Metal stairways, with $1\frac{1}{2}$-in. hand-rail, shall be provided for to the machinery, trunnions, and counterweights. The pipe weigh 2.68 lb. per ft.

Girders in Rolling Bridges.

26.—The reinforcements of webs in the segmental girders and track girders of rolling bridges shall be symmetrical about the center planes of the webs. The center planes of the segmental webs shall coincide with the corresponding center planes of the webs of the track girders. That part of an outstanding leg of an angle, which is beyond the outside face of the upstanding leg, shall not be considered as reinforcement. The width of contact between the segmental webs (including the reinforcements) and the back of the tread plates, shall be equal to the width of the corresponding contact in the track girder.

Coefficients of Friction for Moving Span and Attached Parts.

27.—In calculating the resistances to be overcome by the machinery the resisting forces shall be reduced to a single force acting between the pinion and operating rack, or in the operating cable. In determining this force, the following coefficients shall be used in starting the span, and, except for the stiffness in cables, shall be reduced one-half after motion is begun. (44):

For trunnion friction.....
For rolling friction of bridges having rollers with flanges, or built-up segmental girders..... $\frac{1}{1}$
For stiffness of wire rope per 180° of bending: d = diameter of rope, in inches; D = diameter of sheaves, in inches..... $\frac{d}{3l}$

Coefficients of Friction.

28.—For a solid cast roller, without flanges, in contact with one surface only, the coefficient of rolling friction shall be taken equal to $\frac{15}{1\,000\,r}$, in which r is the radius of the roller, in inches.

29.—In figuring the machinery losses between the operating rack or operating cable and the motor, the following coefficients shall be used:

For the efficiency of any pair of gears, journal friction not included..... 0.1
For journal friction..... 0.0
Losses in a worm gear for an angle of thread of 20° or more..... 0.

30.—For sliding friction between plane surfaces intermittently lubricated, such as guides on tower posts, the coefficient of friction shall be taken equal to 0.08.

Time to Open.

31.—The time to open the bridge after the ends are released shall be approximately as specified on the proposal drawing.

Inertia.

32.—The force necessary to overcome the inertia and produce acceleration and retardation for the time of opening shall be considered. The machinery shall be capable of stopping the bridge in 6 sec.; for this purpose, the coefficient of friction in the friction brake shall be taken at not less than 25 per cent.

Impact in Structural Parts.

33.—In calculating the dead-load stresses in the moving structural parts, for the various positions of the open bridge, such stresses shall be increased 25% as allowance for impact. For stationary structural parts (as towers, and supporting girders in rolling bridges), to which moving structural parts are attached, or on which such parts roll, 15% of the static stress shall be added as impact.

34.—In structural steel parts, where a percentage of the dead load or static stress is added for impact, the unit stresses for stationary structures shall be used; the impact percentages are an allowance similar to that provided by an impact formula for stationary railroad bridges.

Reversal of Stress.

35.—In structural members subject to reversal on account of the motion of the span, the effect of reversal shall be neglected. The member shall be designed for the stress giving the larger section. For riveted connections, the number of rivets shall be increased 25% over that required for the static stress plus impact stress.

Impact for Machinery Parts, etc.

36.—The allowance for impact in trunnions, cables, cable attachments, machinery parts, and structural parts supporting the machinery, is taken care of by lowered unit stresses.

Wind Pressure.

37.—The wind pressure to be assumed in proportioning the machinery or moving parts shall be 15 lb. per sq. ft. on the exposed surfaces of the moving parts as projected on any vertical plane. The machinery shall be strong enough to hold the moving parts in any position for this pressure, and be capable of opening the bridge in the specified time at 10 lb. per sq. ft. wind pressure.

38.—The structure shall be strong enough to resist a wind pressure of 15 lb. per sq. ft. on any surface as projected on any vertical plane for any open position of the moving span. The structure shall be strong enough to resist a wind pressure of 25 lb. per sq. ft. when the moving span is in the closed position.

Least Wind Pressure.

39.—The least wind pressure to be assumed on the floor of the moving span shall be $2\frac{1}{2}$ lb. per sq. ft. for any position of the span. For the vertical lift, this shall be taken as acting throughout the movement.

40.—On the ordinary open-floor bridge with ties, the exposed surface to wind shall be taken equal to 80% of a full quadrilateral having a width equal to the distance from center to center of trusses and a length equal to that of the moving span.

Detailed Drawings.

41.—The Contractor shall make complete detailed drawings of the machinery, so that any other shop can take them and duplicate the machinery. No reference to patterns or individual shop practices will be considered in lieu of the complete drawings. These drawings shall show a general outline of the assembled machinery. The drawings shall be made on tracing cloth, each sheet 24 by 36 in. in outside dimensions. These drawings shall become the property of the Railroad Company on the completion of the job.

Outline Drawing of Machinery.

42.—The Contractor shall furnish an outline drawing of the machinery, on which are shown the forces acting on the gear teeth, the twisting moment and bending moment on shafts, and other necessary information for checking the strength of the machine parts. A tabulation of the formulas and methods of calculation shall be shown complete enough to allow them to be checked.

Torque Curves.

43.—The Contractor shall show by a drawing of curves the torque to be exerted by the motor or prime mover, as follows:

1. A torque curve for acceleration and retardation;
2. A torque curve for the frictional resistance;
3. A torque curve for any unbalanced condition of the structure;
4. A torque curve for the wind load;
5. A torque curve showing the greatest combination of resistances acting at any one time.

44.—In figuring the friction at starting (this being twice the running friction), no acceleration of the moving mass shall be considered. This friction shall be considered as reduced to the running friction in the first second after the power is applied. (27).

Capacity of Wires, etc.

45.—If the Contractor is to furnish the design for the electrical equipment, such as wiring, switches, etc., he shall show by a curve the current required by the motor to overcome the various resistances. This is for the purpose of checking up the carrying capacity of wires and other parts, and determining the storage battery capacity.

Center of Gravity.

46.—The Contractor shall check the location of the center of gravity of the moving span, including all parts attached thereto, and also the location of the center of gravity of the counterweights, including counterweight girders and trusses, by computations based on accurate weights calculated from shop plans. He shall submit duplicate sketches and copies of these computations accompanied by weight bills to the Railroad Company for approval.

Hand Operation.

47.—All bridges shall be equipped with hand-operating mechanism. The number of men and the time required to operate shall be estimated on the assumption that the force one man can exert on a lever is 40 lb. with a speed of 160 ft. per min. developing about $\frac{1}{5}$ h.p. For calculating the strength of the machinery, the power of one man shall be assumed as 125 lb.

Operating Machinery.

48.—The parts shall be simple in design, and easily erected, inspected, adjusted, and taken apart. The fastenings shall hold the parts in place securely after they have been set.

Kind of Material.

49.—Rolled or forged steel shall be used for bolts, nuts, keys, cotters, pins, axles, screws, worms, piston rods, trunnions, and crane hooks, if any.

50.—Trunnions, pins, and shafting more than 4½ in. in diameter, shall be of forged structural steel. Shafting 4½ in. or less in diameter may be of cold-rolled steel.

51.—Forged or cast steel shall be used for levers, cranks, and connecting rods.

52.—Cast steel, or forged steel, shall be used for couplings, end shoes, racks, toothed wheels, brake wheels, drums, sheaves, and hangers where the supported weight will cause tensile stresses. Large sheaves may be built of structural steel.

53.—Pinions shall be made of forged steel and cut from the solid metal, unless such pinions are too large for forgings.

54.—Sockets used for holding the ends of wire ropes shall be forged without welds from the solid steel. The equalizing levers, connecting the ropes to the counterweights, or moving span, shall be of forged steel.

Cast Iron

55.—Cast iron may be used in boxes for shafts 2 in. or less in diameter, and which obviously carry light loads. Other boxes shall be of cast steel.

56.—Cast iron may be used in eccentrics, cylinders, pistons, fly-wheels, and parts of motors which are usually made of cast iron. Cast iron shall not be used for any trunnion or axle supports.

Metal for Bushings

57.—Phosphor-bronze, brass, and Babbitt metal shall be used for the bushing or lining of journal bearings and other rotating or sliding surfaces, to prevent seizing.

58.—Phosphor-bronze, only, shall be used for bushing for the trunnions of bascule and lift bridges, or in any large bearing carrying heavy loads.

59.—The bushings for large bearings, such as for trunnions and similar parts, shall be held from rotating in their casings. The force tending to cause rotation shall be taken as one-eighth of the load on the trunnion or bearing, and as acting at a tangent to the surface between the back of the bushing and casing; this force shall not be considered as counteracted by any frictional resistances between bushing and casing. It shall be practicable to take out the bushing when the trunnion is slightly lifted.

Castings

60.—Castings which are to be attached to rough unfinished surfaces shall be provided with chipping strips. The outer unfinished edges of ribs, bases, etc., shall be rounded off, and inside corners filleted.

Bolts and Nuts.

61.—Bolts and nuts, up to 1½ in. in diameter, shall have U. S. Standard V-threads. Nuts and exposed bolt heads shall be hexagonal in shape, and each nut shall be provided with a washer. If the nut will come on an inclined surface, a special seat, the top surface of which is at right angles to the bolt, shall be cast or built up to receive the nut. Bolt heads which are countersunk in castings shall be square.

62.—Nuts which are subject to vibration and frequent changes of load shall have locking arrangements to prevent the gradual unscrewing of the same. If double nuts are used for that purpose, each nut shall be of the standard thickness. Nuts subject to vibration shall be further secured by split pins through the bolt.

Screws.

63.—Screws which transmit motion shall have square threads.

p-Bolts, Set-crews, etc.

64.—Tap-bolts and stud-bolts shall not be used, except by special permission.

65.—Set-screws shall not be used for transmitting torsion to shafts or axles.

Collars.

66.—Collars shall be used wherever necessary to hold the shaft from moving horizontally. Each collar shall have at least two set-screws at an angle of 120 degrees.

Shaft Couplings.

67.—Shaft couplings, unless of the flexible kind, shall be of the flange type, or split muff with bolt heads and nuts countersunk.

68.—For large shafts, couplings such as are used for rolling mill shafting may be used.

69.—Couplings shall be keyed to shaftings.

Keys.

70.—If practicable, hooked and tapered keys shall be used. The taper shall be $\frac{1}{8}$ in. per ft. The width of the key shall be one-fourth of the diameter of the shaft; the height, at mid-section of the tapered length, shall be three-fourths of the width. The length of the hook, measured parallel to the shaft, shall be equal to the width of the key.

71.—If tapered keys are not practicable, parallel-faced keys of about the above proportions shall be used.

72.—Tapered keys shall bear on top, bottom, and sides; parallel-faced keys shall bear on sides only.

73.—The length of any key shall be not less than that of the hub.

74.—The foregoing dimensions are approximate. The shape of the key shall be such as to have unit stresses in shear and bearing not exceeding those allowable in the table.

75.—If practicable, the keys and grooves shall be made so that the keys may be backed out.

76.—Keys shall be sunk in grooves in both hub and shaft. The depth of a groove shall be such that the bearing will not exceed the allowable unit stress.

et-Screws, etc.

77.—Keys shall be held by set-screws or equivalent means. In vertical shafts, bands clamped about the shaft, or other devices, shall be placed below the key.

Hub.

78.—If practicable, the length of the hub shall be not less than two diameters of the shaft, and its thickness not less than one-third of the diameter of the shaft. The hub shall have a light driving fit on the shaft.

79.—The groove in the hub shall be made on the center line of an arm.

0.—Hubs shall be bored truly at the center of the wheel.

1.—For trunnions and similar parts, which are designed chiefly for ling and bearing, the keys, key-ways, and bolts shall be designed old the trunnions from rotating. The force tending to cause rota- shall be taken at one-fifth of the load on the trunnion, and shall aken as acting at the circumference of the trunnion. Keys in Trunnions.

2.—Journals shall be proportioned to resist, not only the various ses to which they are subjected, without exceeding the permissible and bending stresses, but also to prevent a tendency to heat and . Journals.

3.—Divided journal and trunnion bearings shall be used, and the shall be fastened to the base with turned bolts recessed into the . The nuts and heads shall bear on finished bosses cast on the bear- There shall be $\frac{1}{8}$ in. clearance between the lining of the base the cap or its lining, to allow for expansion.

4.—Steel bearings carrying steel shafts or journals shall be lined bronze or brass. If shafts are 3 in. or less in diameter and of ow motion, Babbitt metal may be used. Bearings of steel on steel moving surfaces will not be allowed. Bushings.

5.—In cast-iron boxes carrying light shafts, no lining is needed. Boxes.

6.—The bearings of shafts shall be placed as near to the points oading as possible.

7.—The foot-steps of vertical shafts shall be of axle or tool steel, shall run on bronze disks.

8.—Provision shall be made for the effective lubrication of jour- , or any other sliding surfaces. Closed oil or compression grease s shall be used. Grooves shall be cut in the surface of the trunnion rovide for the proper distribution of grease or oil. Grease and oil s must hold the lubricant for any position of the moving parts. Lubrication.

9.—The grooves in large trunnions shall approximate to a U shape; size shall be such that a wire $\frac{5}{16}$ in. in diameter may lie wholly in the groove. The edge of the U shall be rounded to a radius in. Grease Grooves.

0.—The grooves shall be straight, running parallel to the axis of trunnion. They shall be not less than three in number, and lo- d so that all parts of the bearing surface of the bushing will be pt by the contained lubricant, in an opening, and in a closing of bridge. The grooves must allow of being cleaned with a wire.

1.—In any trunnion bearing, or similar heavy bearings, strong pression grease cups shall be used for the grooves. Grease Cups.

2.—Oil and grease ducts shall be located so that the lubricant will w by gravity toward the bearing surface.

3.—Dust covers shall be provided for the principal bearings, in rticular for trunnions where shown on plans. Dust Covers.

Shaft Supports and Couplings.

94.—Line shafts, extending from the center of the bridge to t end, shall not be continuous, but shall be connected with claw cou lings. Each length of shafting shall rest in not more than two bea ings, with the couplings close to the bearings.

95.—If shaft supports are connected to the floor-beam, in bridg having long panels, intermediate supports shall be used; these sh be adjustable, and are intended merely to prevent the shaft fr sagging.

Equalizing Gears.

96.—Equalizing gears or devices shall be used to insure equal tion at the pinions and operating racks.

Unsupported Length of Shafts.

97.—The unsupported length of shafts shall not exceed $L = 80^3 \sqrt{}$ for shafts supporting their own weight only; $L = 50^3 \sqrt{d^2}$ for sha carrying pulleys, gearing, etc., where L = length of shaft betwe center of bearings, in inches; and d = diameter of shaft, in inches.

98.—Line shafts connecting machinery at the center to that at t ends shall run at fairly high speed. The speed reduction shall be ma in the machinery near the end.

99.—In designing circular shafting, trunnions, and axles, the gre est unit fiber stress in tension or compression due to bending shall calculated by the following formula:

$$f = \frac{32}{\pi d^3}\left(\frac{3}{8}M + \frac{5}{8}\sqrt{M^2 + T^2}\right).$$

Formulas for Shafts.

100.—The maximum unit shear shall be calculated by the follow formula:

$$S = \frac{16}{\pi d^3}\sqrt{M^2 + T^2}.$$

101.—In these formulas, f = unit fiber stress in tension or c pression; S = unit shear; d = diameter of shaft; M = the sim bending moment; and T = the simple twisting moment.

102.—If a shaft, trunnion, or axle has one key-way cut, at the tion where the maximum stresses occur, f and S shall be increased one-sixth; if two key-ways are cut, increase by one-fourth. If shaft is enlarged through the hub, this does not apply.

Distance between Shaft Supports.

103.—In calculating the bending moment on shafts, trunnions, journals, the distance from center to center of bearings shall be ta

Style of Gear Teeth.

104.—Gear teeth shall be of the involute type, with an angle obliquity of 20 degrees. The roots below the clearance line shall filleted.

105.—The width of the teeth may be as great as four time te pitch, but not more, except for wheels running at a very high veloci as in motors where abrasion is to be considered.

Strength of Beveled Gear Teeth.

106.—In estimating the strength of teeth in bevel wheels, the pi at the middle section shall be taken.

Pitch Circl

107.—For the purpose of setting gear teeth accurately in the field erection, the pitch circle shall be scribed on the ends of the teeth.

Worm Gearing.

108.—Worm gearing, for transmitting power, shall have an angle of thread not less than 20 degrees. The worm shall run in oil. A bronze or brass collar shall be used at the end of the worm and at the end of the wheel axle, to take care of the end thrust. The wheel shall be of bronze. If a nut engages the worm, the nut shall be of bronze.

109.—Worms which are to be used for actuating signals, indicators, or other minor parts, may have an angle of thread less than 20 degrees.

Number o Teeth in Worm Gear

110.—Worm wheels shall have not less than twenty-six teeth.

Number o Pinion Teet

111.—Pinions shall have not less than fifteen teeth.

Diameter o Sheaves.

112.—For the purpose of keeping down the wear between the individual wires in counterweight rope, the diameter of the sheave shall be at least ninety times that of the rope.

Safety Guar

113.—Machinery parts near which workmen may be while the parts are in motion, shall be designed so that safety guards may be added. These guards will be furnished and installed by the Railroad Company.

Counterbalancing, Operating Ropes, and Attachments.

Wire Rop and Cable

114.—Wire rope shall be made by a manufacturer approved by the Engineer.

115.—The counterbalance ropes shall be of plow-steel wire, and shall consist of six strands, of nineteen wires each, laid around a hemp center.

116.—Ropes shall be laid up in the best manner, and shall be thoroughly soaked in an approved lubricant during the process of manufacture.

117.—The counterbalance ropes shall be made from wire which has been tested in the presence of an inspector designated by the Engineer, and which, for sizes from 0.076 to 0.150 in. in diameter (the limiting values used in counterbalance ropes), exhibits the following physical properties:

a.—The tensile strength shall be not less than 225 000 lb. per sq. in. for wire from 0.150 to 0.126 in. in diameter, nor less than 230 000 lb. for wire from 0.125 to 0.101 in. in diameter, nor less than 235 000 lb. for wire from 0.100 to 0.076 in. in diameter.

b.—The total ultimate elongation, measured on a piece 12 in. long, shall be not less than 2.4 per cent.

c.—The number of times a piece 6 in. long can be twisted around its longitudinal axis without rupture shall be not less than 1.4 divided by the diameter, in inches.

d.—The number of times the wire can be bent 90°, alternately to the right and to the left, over a radius equal to twice its diameter, without fracture, shall be not less than six. This test shall be made

in a mechanical bender constructed so that the wire actually conforms to the radius of the jaws and is subjected to as little tensile stress as possible.

Ultimate strength of Cables.

118.—The rope shall be made in one piece, if possible. Its breaking strength, as determined by test described in Paragraph 121, shall be not less than

4 900	lb.	if	$\frac{1}{4}$	in.	in	diameter.
11 800	"	"	$\frac{3}{8}$	"	"	"
20 600	"	"	$\frac{1}{2}$	"	"	"
32 400	"	"	$\frac{5}{8}$	"	"	"
45 000	"	"	$\frac{3}{4}$	"	"	"
70 000	"	"	$\frac{7}{8}$	"	"	"
79 200	"	"	1	"	"	"
100 800	"	"	$1\frac{1}{8}$	"	"	"
120 600	"	"	$1\frac{1}{4}$	"	"	"
148 000	"	"	$1\frac{3}{8}$	"	"	"
173 000	"	"	$1\frac{1}{2}$	"	"	"
200 000	"	"	$1\frac{5}{8}$	"	"	"
230 000	"	"	$1\frac{3}{4}$	"	"	"
264 000	"	"	$1\frac{7}{8}$	"	"	"
297 000	"	"	2	"	"	"
325 000	"	"	$2\frac{1}{8}$	"	"	"
374 000	"	"	$2\frac{1}{4}$	"	"	"
465 000	"	"	$2\frac{1}{2}$	"	"	"

119.—In case the physical qualities of the rope, or its individual wires, fall below the values cited above, the entire length from which the test pieces were taken shall be replaced by the manufacturer with a new length, the physical qualities of which come up to the specifications.

120.—The dimensions of the sockets shall be such that no part under tension shall be loaded higher than 65 000 lb. per sq. in. when the rope is stressed to its ultimate strength, as named above. The sockets must be attached to the rope by a method which is reliable and will not permit the rope to slip in its attachment to the socket.

121.—In order to show the strength of the rope and fastenings, a number of test pieces, not more than 10% of the total number of finished lengths which will ultimately be made, nor less than two from each original long length, and not more than 12 ft. long, shall be cut, and shall have sockets, selected at random from those which are to be used in filling the order, attached to each end. These test pieces are to be stressed to destruction in a suitable testing machine. Under this stress the rope must develop the ultimate strength given in Paragraph 118.

122.—If, in testing, slipping in the sockets should occur, then the hod must be changed until slipping is avoided. The sockets themes shall be stronger than the rope with which they are used; if should break during the test, then two others shall be selected attached to another piece of rope and the test repeated; and this cess shall be continued until the inspector is satisfied of their ability, in which case the lot shall be accepted. If, however, 10% nore of all the sockets tested break at a load less than the minimum mate strength of the rope, as given in Paragraph 118, then the re lot shall be rejected.

Length of Rope.

123.—The length of each rope, from inside of bearing to inside of ring of socket, shall be determined, and a metal tag having the length stamped thereon shall be attached securely to the rope.

Testing Rope Connections.

124.—One-third of the wire rope connections, selected at random, ll be tested (after attachment to the ropes are made) up to 0.4 of ultimate strength of the rope. If any connection is weak, the render of the connections shall be tested. The weak connections ll be rejected and replaced. Not less than four connections shall tested.

Facilities for Testing Rope.

125.—The manufacturer shall provide proper facilities for making tests, and shall make at his own expense all the tests required. sts shall be made in the presence of an inspector who represents the gineer.

Shipment of Rope in Coils.

126.—Ropes shall be shipped in coils the minimum diameter of ich is at least thirty times that of the ropes, and they shall be ued for use by revolving the coil, not by pulling the rope away from stationary coil.

Workmanship.

127.—For the parts of the operating machinery of movable bridges ich are usually exposed to the weather, the finish shall be confined the bearing, rotating, and sliding surfaces, and wherever it is red to produce accurate fits and precise dimensions.

128.—Equalizing levers in rope connections shall be neatly finished, shall conform to the dimensions shown on the drawings.

129.—Castings shall be cleaned, and seams and other blemishes noved.

130.—Drainage holes, not less than $\frac{3}{4}$ in. in diameter, shall be lled in places where water is likely to collect.

Play of Unfinished Bolts.

131.—Unfinished bolts may have a play of $\frac{1}{16}$ in. in the bolt holes. rned bolts must have the diameter of the shank at least $\frac{1}{16}$ in. larger an the diameter of the threaded portion, and must have a driving fit the bolt hole.

Racks and Contact Surfaces.

132.—The backs of racks and surfaces in contact shall be planed.

Sheaves. 133.—The bearing surfaces, at the junction of the cast rim an structural parts in built-up sheaves, shall be planed to give full con tact throughout the circumference of the sheave.

Grooves in Sheaves. 134.—The grooves in the circumference of sheaves carrying wir ropes shall be turned to a radius that will fit the rope. This is to b done after the sheave is completely assembled and permanentl riveted up.

Tread-Plates. 135.—The top and bottom of the tread-plates and surfaces i contact in rolling bridges shall be planed to fit. A full bearing mus be made.

136.—The periphery and the ends of teeth which mesh wit shrouded teeth shall be planed, and the pitch line shall be scribe thereon.

Finishing of Trunnions, etc. 137.—Journals and trunnions shall be turned with a fillet where th section changes. Journals shall have a collar at each end. Trunnion and journals 8 in. in diameter and greater shall have a hole, 1½ i in diameter, bored through on the longitudinal axis. Journals, trun nions, and bushings must be polished after being turned. The us of a cutter which trembles or chatters will not be allowed.

138.—The joints between the caps and bases of journal and trunnio bearings shall be planed. The ends of the bases and surfaces in con tact with the supports shall be planed. Bolt holes for holding the ca to the base and for holding the base to its support shall be drilled.

Grooves. 139.—The grooves in the surfaces of trunnions or similar lar bearings shall be machine cut. Chipping and filing will be allowe only for removing small inequalities. The grooves shall be smoot especially the rounded corners.

Hubs. 140.—Hubs of wheels, pulleys, couplings, etc., shall be bored to close on the shaft axle. If the hub performs the function of a coll the end next to the bearing shall be faced. Holes in hubs of tooth gear wheels shall be concentric with the pitch circle.

Cut Gears, etc. 141.—The periphery of gear wheels shall be turned. Gear whe and racks which are part of the train which actuates the movi span shall be cut. Other gears, except beveled gears, shall be machi moulded.

142.—If any moulded gears are shrouded, chipping or other me shall be used at the junction of the shrouding and teeth to ins proper meshing.

Beveled Gears. 143.—Beveled gears shall be cut. The cutting shall be done by planer having a rectilinear motion to and from the apex of the co Rotating milling cutters shall not be used.

144.—Threads on worms, and the teeth of worm wheels shall cut, and shall fit accurately. Point contact shall be avoided.

145.—Any two surfaces which slide, or roll, or bear on each other, ll be planed or turned to fit.

Assembling of Machinery.

146.—Machinery parts shall be assembled on the supporting mem- s in the shop, and shall be aligned and fitted, and holes in the sup- ts drilled, and with the members in correct relative position. The nbers shall be match-marked, both to the supports and to each other, re-erected in the same relative position.

Holes for Sheaves for Vertical Lift Bridges.

147.—The holes in the girders and columns for the bolts connecting main sheave bearings to their supporting girders shall be drilled n the solid through cast-iron or steel templates on which the bear- s were set and accurately lined when the holes in the bearing were ed. The bolt holes and the bolts shall be turned to the same diame- and the bolts shall be driven to place without injury to them, the rings, the girders, or the columns.

Testing of Trunnion Bearings.

148.—Trunnions shall be turned to a diameter of $\frac{1}{50}$ in. less than of the bushing. Before shipping, the trunnions shall be placed their bearings and given two full rotations. If any grinding or d turning is found, it must be remedied. The tests shall be made he presence of the Railroad Company's inspector.

Facing of Couplings.

149.—Faces of flange and split muff couplings shall be planed to fit. couplings shall be keyed to the shaft.

150.—A special effort to secure good workmanship on keys and key- s shall be made.

Coating of Surfaces.

151.—Machined surfaces shall have a coating of white lead applied hem.

152.—Machinery which is of the regular standard manufactured e, such as steam, gasoline, electric motors, pumps, air compressors, shall be guaranteed by the manufacturer as to efficiency, and shall subject to the approval of the Engineer. Motors shall be tested to ve that they fulfill the specified requirements and develop the de- d speed, power, and torque.

153.—The rating of a prime mover shall be the horse-power deter- ed by the brake test.

Unit Stresses.

Normal and Excess Loads.

154.—Machinery parts shall be designed for the normal loads used determining the torque curves, using the unit stresses herein speci- . For the excess torque specified for the prime mover, twice the mal unit stresses will be allowed. (178) (205) (208).

Braking.

155.—If brakes act through the machinery, the unit stresses pro- ed by braking shall not exceed by more than 50% those caused by normal torque of the prime mover.

156.—The following unit stresses, in pounds per square inch, shall used for parts in which main stresses are not increased by impact:

Stresses in One Direction.

Material.	Tension.	Compression.	Fixed Bearing.	Shear
Machinery steel...	9 400	$9\,400 - 40\,\frac{l}{r}$	11 000	6 200
Structural steel....	8 500	$8\,500 - 36\,\frac{l}{r}$	.....	5 600
Steel castings.....	7 000	$8\,000 - 35\,\frac{l}{r}$	.....	5 000
Phosphor-bronze..	6 600	.....	.....	4 600
Cast iron.....	3 000	8 000	.....	3 000
	Shear on keys....	4 900		
	Bearing on keys..	8 800		

157.—The maximum unit tension in plow-steel cables shall be one-sixth of the ultimate. The maximum unit tension is equal to the direct unit stress plus the extreme fiber unit stress in the individual wire due to bending over the sheave.

Reversal of Stresses.

158.—For stresses which are reversed at the rate of ten or more times per minute, use one-half of the above unit stresses.

159.—If wire rope is bent over a sheave, the bending stress and permissible load on the rope shall be calculated as follows:

Let P = the total pull or permissible load on the rope, in pounds;
K = extreme unit fiber stress in greatest individual wire;
E = modulus of elasticity = 28 500 000;
a = cross-sectional area of rope, in square inches;
d = diameter of thickest wire, in inches;
D = diameter of sheave to center of rope, in inches;
S = greatest unit tension allowable;
α = angle of helical wire with axis of strand;
β = angle of helical strand with axis of rope;
c = diameter of rope.

$$\text{Then } K = \frac{Ed \cos.^2 \alpha \cos.^2 \beta}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$P = a \left(S - \frac{Ed \cos.^2 \alpha \cos.^2 \beta}{D} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

For rope having six strands of nineteen equal wires each,

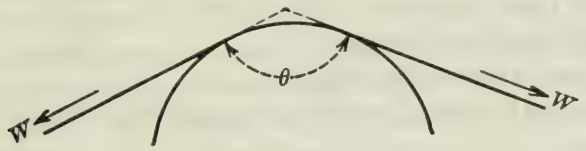
$$P = a \left(S - \frac{1\,800\,000\, c}{D} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

because $\cos.^2 \alpha \cos.^2 \beta = 0.95$, $d = \frac{c}{15}$.

160.—For haulage rope, six strands of seven wires each, take $d = \frac{c}{9}$.

161.—If a rope is in contact with a sheave over a small arc, the actual radius of curvature may be greater than that of the sheave. (See figure.)

Let R = The actual radius of curvature;
θ = the angle between the directions of the rope;
W = pull on individual wire, equal to P divided by the number of wires if all wires are of equal diameter.



$$\text{Then } R = \frac{4\,d^2}{17 \cos. \frac{\theta}{2}} \sqrt{\frac{E}{W}}.$$

162.—If R is greater than the radius of the sheave, $2R$ should be used in place of D in Formulas (1), (2), and (3). The formula is only valid for values of θ between 130 and 180 degrees.

Strength of Gear Teeth.

163.—The strength of cut gear teeth shall conform to the following formula, one tooth only taking pressure:

$$P = fpb \left(0.154 - \frac{0.912}{n}\right) \frac{600}{600 + v}, \text{ in which}$$

P = pressure on tooth, in pounds;
f = permissible unit stress = 17 000 lb.;
p = pitch, in inches;
b = face or breadth of tooth, in inches;
n = number of teeth in gear;
v = velocity on pitch circle, in feet per minute.

164.—The strength of machine-moulded teeth shall be calculated by the foregoing formula, taking f = 15 000 lb.

165.—The foregoing formula is for involute teeth having an angle of obliquity equal to 20 degrees.

Pressure on Rollers.

166.—The pressure, in pounds per linear inch, on rollers at rest shall be, for rolled and cast steel, 600 d, where d equals the diameter of the roller, in inches.

Unit Stresses for Bearing on Rotating and Sliding Surfaces.

167.—Maximum bearing values for rotating and sliding surfaces, in pounds per square inch:

Bearings on which the speed is slow and intermittent:

	Pounds per square inch.
Trunnion bearings on bascule bridges: machinery steel on phosphor-bronze	1 500
Wedges: Cast steel on cast steel or structural steel	500
Screws which transmit motion on projected area of thread	200

	Pounds per square inch.
For ordinary cases, parts moving at moderate speeds: Hardened steel on hardened steel	2 000
Hardened steel on bronze	1 500
Tool steel (not hardened) on bronze	900
Structural steel on bronze	600
Cast iron on structural steel	400
On cross-head slides, speed not exceeding 600 ft. per min	50

168.—In order to prevent heating and seizing at higher speeds, the pressure on pivots or foot-step bearings for vertical shafts and journals shall not exceed:

On pivots ..... $p = \frac{40\,000}{n\,d}$ per square inch.

On journals ..... $p = \frac{300\,000}{n\,d}$ per square inch.

Where n = number of revolutions per minute;
d = diameter of journal or pivot, in inches.

169.—For crank pins and similar joints with alternating motion, the limiting bearing values given in the above formula may be doubled.

resses on Rollers.

170.—Permissible pressure, in pounds per linear inch of roller in motion:

For cast iron	$p = 200d$
For steel castings	$p = 400d$
For machinery steel	$p = 500d$
For tool steel	$p = 800d$
For hardened tool steel	$p = 1\,000d$

Where p = pressure per linear inch of roller,
and d = diameter of roller, in inches.

171.—The foregoing values are for rollers and bearing surfaces of the same material; if rollers and bearing surfaces are of different materials, the lower value shall be used.

Power Equipment.

General Requirements.

172.—The kind of motor best adapted to any particular case depends on local conditions, and should be left to the judgment of the Engineer.

echanical Power.

173.—If the bridge is operated by mechanical power, the motor shall be of ample capacity to move or turn the bridge at the required speed. No matter what mechanical power is used, all bridges shall also be provided with hand-power operating machinery.

Friction Brakes.

174.—Friction brakes, to be operated by hand or foot, shall be provided where the motor is located in the operator's house. They shall have sufficient capacity to stop or hold the moving span in any position, under all conditions.

Operator's House.

175.—If mechanical power of any kind is to be used for operating a movable bridge, a suitable house shall be provided for the operator. The house shall be of such dimensions as required for the purpose for which it is to be used. It shall be placed in a position where the operator can observe the signals and see the approaching vessels and trains, and with enough windows of sufficient size, so that his view will not be obstructed. If the operator's house is above or below the floor of the bridge, suitable steel or iron stairs with railings shall be provided to lead from the floor of the bridge to the floor of the operating house. The house shall be of fire-proof construction, consisting of a steel frame, steel floor joists and a fire-proof floor. If the house contains motors and machinery, the floor shall preferably consist of steel plates, but, if the motors are located elsewhere, the floor between the joists may be of concrete construction. The sides and roof shall be of metal, concrete, or any other non-combustible material. The hand-rail for stairways and other places shall be made of $1\frac{1}{2}$-in. gas pipe.

Heating of Operator's House.

176.—Whenever climatic conditions require it, provision shall be made for heating the operator's house. If steam power is used, the house shall be heated by a steam coil or radiator fed from the boiler. If electric power is used, the heat may be supplied by electricity. If gasoline is used, or any other power which cannot be utilized for heating, a coal, wood, petroleum, or gas stove, as directed by the Engineer, shall be provided.

Whistle.

177.—A whistle, having a bell 5 in. in diameter and 12 in. long, shall be installed complete. If operated by air, the compressor and air tank shall conform to the following specifications: The compressor shall be motor driven, the motor and compressor being on one frame, and geared. All working parts shall be completely enclosed, and self-lubricating. The compressor shall have a piston displacement of from 25 to 30 cu. ft. per min. when working against a tank pressure of 90 lb. per sq. in. The compressor shall be provided with strainer, and automatic governor and switch, in order that the compressor may start and stop automatically at any predetermined tank pressure. The air receiving tank shall be 36 in. in diameter and 8 ft. long, or of equal capacity. The tank shall be galvanized, and good for a working pressure of 100 lb. per sq. in. It shall be provided with pressure gauge and pigtail, pop-valve and drain cock, and have standard flanges bushed for $1\frac{1}{2}$-in. pipe. The Contractor shall furnish all pipe, pipe fittings, and valves, and all shall withstand a working pressure of 100 lb. per sq. in.

Greatest Torque.

178.—A prime mover shall be capable of exerting twice the greatest torque shown on the torque curves for the normal loads. (154).

Steam Power.

Steam Engine. 179.—If a steam engine is used, it shall consist of a double-cylinder, reversing engine, the piston speed of which shall not exceed 200 ft. per min.; it shall develop the desired power and speed with a steam pressure of 50 lb. per sq. in. The engine shall be connected to the operating machinery by an approved friction clutch, arranged so that the moving and locking machinery can be operated alternately or stopped without stopping the engine.

Steam Separator. 180.—In the steam supply pipe, and close to the steam chest, shall be placed a steam separator. This separator, under test with quality of steam as low as 66%, shall show an average efficiency of 85% in five tests.

Boilers. 181.—The steam shall be generated by one or two upright, tubular boilers, each of which shall have twice the capacity of the engine. The boilers shall be designed for a steam pressure of 150 lb. per sq. in., and adapted to the kind of fuel specified by the Engineer; they shall be of open-hearth steel, in accordance with the specifications for boiler plates appended hereto. They shall be encased in asbestos and covered with Russia iron.

182.—The boilers shall also be in accordance with the specifications of the Mechanical Department of the Railroad Company, and shall conform to the civil laws.

Flues of Boilers. 183.—Vertical boilers shall have submerged flues at the top.

Horse-power of Boilers 184.—The total horse-power of the boilers shall be twice that of the engine, and shall be computed by the following rule: Calculate the inside area of the tubes, the area of the tube sheet next to the fire, and the sides of the fire-box where this is in contact with the fire. Take the sum of these areas in square feet and divide by fifteen. The intention is to allow 15 sq. ft. of heating surface per horse-power. At least $\frac{1}{3}$ sq. ft. of grate surface shall be provided per horse-power.

Equipment of Engine-Room. 185.—The engine-room shall be provided with a steel water tank of sufficient capacity; a duplex, steam feed-pump; and an injector for each boiler, with necessary pipes and connections for feeding boilers separately or together; steam water-lifters with necessary strainers, flexible hose, and piping to lift the water from the river into the tank; a coal hoist and a steel coal-bin of sufficient capacity. The engine-room shall be provided with a suitable indicator for recording the positions of the moving span in turning and locking.

Internal-Combustion Engines.

Gasoline Motor, etc. 186.—If an internal-combustion engine is used, one of the most substantial kind shall be selected, the maximum piston speed of which shall not exceed 500 ft. per min. The engine shall have a reversing gear provided with approved friction clutches, to be operated by a hand-wheel. The countershaft connecting the engine with the operat-

ng machinery shall be provided with disengaging couplings, arranged so that the moving and locking machinery can be operated alternately and in either direction without stopping the engine. Motors of 10 h.p. or more shall be started by compressed air. The fuel tank shall be located outside of the engine-house. The engine-room shall be provided with indicators for recording the positions of the moving span, and lifting and locking apparatus.

Engine Cooling.

187.—For bridges which are to be opened at intervals of 10 min. or less, and about six times per hour, the engine shall be water-cooled. For longer intervals, the engine shall be air-cooled. For this purpose, the outside cylinder shall have deep flanges about which a forced circulation of air is maintained by a fan.

Ignition.

188.—The ignition shall be of the jump-spark kind in which the secondary coil is made up on each spark plug as part of it, so that a low voltage current of not more than 10 volts will be sufficient.

Extra Parts.

189.—Two extra igniters and two extra crank-pin brasses shall be furnished.

Electric Equipment.

A. I. E. E. Rules.

190.—The electric equipment shall conform to the Standardization rules of the American Institute of Electrical Engineers, as adopted June 21st, 1907, or subsequent revisions.

N. E. C.

191.—The National Electric Code shall apply to the electric construction and installation, except as may be noted hereinafter.

Wires and Insulation.

192.—The quality of the wires and insulation shall conform to the specifications of the Railway Signal Association, as revised and adopted October, 1911. (See Volume 8 of the *Proceedings* of that Association.)

Tests.

193.—Any motor under test shall develop the required horse-power and torque at the armature shaft. Characteristic curves showing the results of the test shall be furnished by the manufacturer.

194.—Motors shall be tested for one-half their normal voltage between the brushes. An efficiency curve shall be shown.

195.—Motors, generators, automatic circuit breakers, solenoids, brakes, and other electric mechanism shall be tested at the factory by the manufacturer in the presence of the Railroad Company's inspector.

196.—If the motor is enclosed in a case, as mill motors are, small openings of sufficient size shall be provided in the case for the inspection, removal, and replacing of brushes.

197.—One cast-steel cut gear, bored and key-seated for attachment to the countershaft, shall be furnished with the motor. The gear and pinion shall be covered by a sheet-steel or malleable-iron split gear case, supported by the motor frame and completely covering the gear and pinion. An opening, with a hinged cover, shall be provided in

the gear case for inspection and oiling. The gear ratio shall be such that the full speed of the countershaft will not be more than 125 rev. per min.

Motor Pinions.

198.—Motors shall have a forged-steel cut pinion, out of one piece, keyed to the end of the armature shaft and secured by a lock-nut.

Spare Motor Parts.

199.—For each size of motor furnished, the Contractor shall supply the following spare parts: One armature, one field coil, one pinion, one gear, and one set of brushes. These parts shall be finished and fitted in such a manner as to admit of being installed in their respective places without further fitting or adjustment.

Mounting Motors.

200.—The motors shall be mounted in such a manner as to admit of easy access for inspection and repairs; they shall be supported securely by brackets or suitable foundations.

201.—If the machinery and motors are on the moving span, they shall be capable of being operated satisfactorily in any position of the span.

Housing of Motors.

202.—Motors must be housed in weather-proof metal housing. This housing must be large enough to allow the inspection and oiling of the motor. It must be readily removable so that access to the motor may be obtained. No metal in this housing shall be less than No. 16, U. S. Standard, gauge; it shall be galvanized.

D. C. Motor.

203.—Direct-current motors and generators shall be of the railway series, or mill, interpole type, water-proof, with slotted-drum armature and form-wound armature coils. They shall be of a standard commercial type in common use.

Testing of Motors.

204.—The rating of a direct-current motor is the horse-power output at the armature shaft which gives a rise of temperature above the surrounding air (referred to a room temperature of 25° cent.) not exceeding 90° cent. at the commutator and 75° cent. at any other part after a continuous run of 1 hour at its rated voltage, on a stand with the motor covers removed and with natural ventilation. The rise in temperature is to be determined by thermometer, but the resistance of no electric circuit in the motor shall increase more than 40% during the test.

Excess Motor Loads.

205.—Direct-current motors shall be capable of exerting continuously for two cycles twice the normal torque shown on the torque curves for the moving span and machinery. One cycle is an opening and closing of the bridge in a specified time. (154) (224) (226).

A. C. Motors.

206.—Alternating-current motors shall be of the three-phase, induction type, with slip-rings, rotor-wound, 25 or 60 cycles, and 220 or 440 voltage, unless otherwise specified. The resistance for varying the speed shall be in series with the rotor circuit, and be such as to affect evenly all three phases. Motors of 5 h.p. or less may be of the squirrel-cage type.

207.—Alternating-current motors shall show, in a run for heat test, the following maximum temperature rises above 25° cent. for

he surrounding room: for continuous run under a nominal load, 0° cent.; for 2 hours' run under 25% overload and a 1 min. run nder 50% overload 55° cent.

208.—Alternating-current motors shall be of rugged construction. The sum of the starting torques of the motors shall be at least equal o twice the greatest torque shown by the torque curves for the bridge perating machinery. The pull-out torque shall be at least equal to 4 times the starting torque. (154) (224) (226).

Controllers.

209.—The controllers for motors shall be located in the operating-ouse. The controllers shall be of the reversing drum type, or flat ype, with magnetic blow-out, and shall be capable of varying and naintaining the speed of the motors throughout the entire range de-red, without injurious sparking, and without shock due to sudden ariation in speed. The controllers shall be capable of doing their ork for the usual loads, and excess loads, that may come upon the notors, with a temperature rise not exceeding that specified for the notors.

Controller Steps.

210.—The controller shall have a sufficient number of notches or teps, such that the minimum or maximum motor torque will not iffer by more than 10% from the average torque required for uni-orm acceleration.

211.—One controller shall be furnished for the operation of the nain motors, one for rail-lock motors, and one for bridge-lock motor. These controllers shall be designed so that the operation of any motor an be cut out by pulling a switch on the switch-board, without affect-ng the operation of any of the other motors.

212.—The controllers for the two main motors, if for direct cur-ent, shall be of the series-parallel type; or of the type in which the eld is varied, as may be done for the interpole type of motor.

Control of Motors.

213.—The control of motors shall be electrically interlocked with ach other and with the signal system, and the bridge shall be con-rolled in such a way that the end-locks cannot be released until the gnals have gone to danger position and the derails are set; and the ridge motor cannot be started until the end-locks have actually been eleased. In closing the bridge, the control shall be such as to make impossible for the operator to move the end-locks until the bridge as been completely closed, or to set the signals at safety until the ridge has been closed and the end-locks are in place.

Master Controller.

214.—For currents too large for the usual type of controller, the notor circuits shall be made by contactors mounted on panels or rames. These contactors shall be operated by solenoids which are ontrolled by a master controller.

Automatic Control.

215.—For large structures, automatic control may be used, but this too complicated to be covered by a specification. This should be aken up for special consideration with the Engineer.

Resistance.

216.—Resistances shall be of the cast-grid type, and of such capacity that the motor can be operated continuously at any point of the controller when developing full-load torque, or for 10 min. when developing 50% over-load torque, without sufficient rise in temperature of the resistance to cause deterioration of any part. The resistance shall be mounted so as to admit of free ventilation and be without injurious vibration.

Electric Brakes.

217.—The main operating motors, rail-lock motors and bridge-lock motors shall be provided with approved post brakes which are held in set position by a spring with such force as to overcome not less than 50% of the maximum torque required. The friction surfaces are to be of materials not affected by moisture. The brakes are to be released by solenoids of ample power and heating capacity whenever the motors are taking current, and are to be set automatically whenever the current fails or is cut off from the motors. Water-proof motors shall be provided with water-proof solenoids. Brakes shall be provided with a foot-switch release for coasting purposes. Means shall be provided for mechanically releasing the brakes when the bridge is to be operated by hand or other equipment.

Emergency Brakes.

218.—An additional emergency brake shall be provided and applied to the main operating machinery. This shall be released by solenoids or motors which shall hold the brake in release as long as the current is applied to the brake motor. Cutting off the current from the solenoids or motors, or any failure of current, will result in the instantaneous application of the brake. This brake will be normally set, but will be released by the operator before starting the bridge, and be held in release during the entire operation unless an emergency condition arises requiring brake power in excess of that offered by the motor brakes, in which case it may be instantly applied by the operator. After the bridge has been closed and traffic has been resumed, this brake will again be applied. This portion of the equipment shall be designed so that it will not be injured if left in release indefinitely. Proper means shall be provided for releasing the brake mechanically when the bridge is to be operated by hand or emergency-power equipment.

219.—The emergency brake circuit shall be independent of the general interlocking system, and there shall be a mechanical interlocking device which will prevent the main leaf motors and the emergency brake from being used one against the other.

220.—The emergency brake switch shall be attached to the controller stand within easy reach of the operator, and proper labels shall be placed back of the switch handle to indicate "Set" and "Released" positions of the brake.

Automatic Cut-offs.

221.—An automatic cut-off or short-circuiting device shall be provided which will throw out the circuit breakers, cut off the current

from the operating motors, and set their brakes when the bridge is 5° from its open position, and its closed position. Spring switches shall be provided which, if closed and held closed, will put the cut-offs out of commission and thus enable the bridge tender fully to close or open the bridge.

222.—The bridge-lock motors, and rail-lock motors, shall be stopped and the brakes set automatically at each end of the travel.

Size of Switches.

223.—Switches shall be designed to carry not more than 900 amperes per sq. in. of cross-sectional capacity.

Excess Current.

224.—Electrical parts, such as wires, switches, etc., shall be designed for the currents which are needed for the excess torques in motors. (205) (208).

225.—Ground connections of ample area shall be provided.

226.—Circuit breakers and fuses shall be designed to act when the current exceeds $2\frac{1}{4}$ times the current required in the motors to exert their greatest normal torque and horse-power for the specified time. (205) (208).

Fuses.

227.—Enclosed fuses shall be used.

Minimum Stranded Wire.

228.—No wire smaller than No. 10, B. & S. gauge, stranded wire, shall be used.

Wires to be Tagged.

229.—Wires when installed shall be permanently tagged and numbered so that any wire can be traced from the switch-board to the motors, and to the source of power.

Lightning Arrester.

230.—The feeders shall be protected by a pole-switch fuse and lightning arrester mounted on a non-combustible and non-absorbent insulating base.

Quick-Break Switch and Switch-board.

231.—A switch, of the quick-break type, shall be provided for each supply wire. Each motor circuit and each light, signal, indicator, or other circuit, shall be provided with switches which are approved by the Railroad Company's Engineer. The switches shall be mounted on an enameled slate panel switch-board (not less than $1\frac{1}{4}$ in. thick, and free from metallic veins, or flaws) in the operator's house. The switch-board shall be large enough to carry the meters, switches, cut-outs, fuses, etc. Switches, cut-outs, buttons, etc., shall be provided with plates designating their use.

Automatic Circuit Breaker.

232.—An automatic circuit breaker shall be placed on the switch-board in the operating motor circuit of the bridge. Each line to the motor, each line to the electric brakes, and each lighting, signal, indicator, or other circuit, shall be protected by enclosed fuses.

233.—Automatic circuit breakers shall be placed near the top of the switch-board, with the other instruments below.

234.—Any circuit whatsoever shall be protected by fuses, circuit breakers, or equivalent devices, which will insure the excessive current being cut off before any parts are damaged.

Lightning Arresters.

235.—Lightning arresters shall be placed as near as practicable to the parts to be protected, and away from combustible material. A No. 4, B. & S. gauge, wire should be used for the connections; this wire should run in a straight line to a ground plate, and not be connected to any structural parts. To avoid inductive resistances, the wire should not run through a conduit. If a choke-coil is used, it should be thoroughly insulated from the ground and other conductors.

Short Circuiting.

236.—The connections of parts in contact with track shall be such as to allow no short circuiting of track signals.

Protection of Electric Contacts.

237.—Electric contacts shall be protected from the weather or accumulations of dirt.

Coils.

238.—Coils shall be impregnated.

Solenoids, etc.

239.—Solenoids and electrically-operated brakes shall be housed.

Indicators.

240.—The Contractor shall provide and install electric light indicators for the purpose of showing the operator the various positions of the bridge, especially the fully open, entirely closed, nearly open, and nearly closed positions of the bridge, and fully closed and fully open positions of the rail-lock and bridge-locks.

Volt Meter, etc.

241.—A volt meter, ammeter, and watt meter shall be provided on the switch-board.

Ground Detector.

242.—The switch-board shall be furnished with one 2-c.p. lamp for detecting ground, and a 2-c.p. lamp for illuminating the ammeter and volt meter scales.

Lamps for Lighting.

243.—In the operator's house shall be placed ten 16-c.p. lights, and additional lights about the machinery and such other lights as the Engineer may direct. For all lights in the house above ten in number, the Railroad Company will pay the regular market price or furnish them to the Contractor.

244.—Lights of 16-c.p. shall be placed outside at the head and foot of stairways or similar paths.

245.—All lights in the house shall have tungsten filaments. Outside lights shall have weather-proof sockets.

Channel Lights.

246.—The Contractor shall furnish warning and channel lights and signals, in accordance with the U. S. Government requirements, or other harbor requirements. The Railroad Company will furnish a copy of the U. S. Government regulations.

Railroad Signal System.

247.—The Company will furnish and install the railroad signal system, also the master lever and all necessary devices controlling the interlock between this signal system and the bridge as a whole. The Contractor shall furnish and install the necessary devices for interlocking the various parts of the bridge with each other and for connections to the Company's master lever.

248.—Emergency switches shall be provided which will free the various motors from the interlocking, in emergencies. These switches shall be mounted on the switch-board, but covered by a sealed glass case.

Current Supply.

249.—Unless the current supply is taken from more than one ource, it shall be conducted to the switch-board in two independent onductors, one for the supply, and one for the return current.

Submarine Cables.

250.—Submarine cables, if needed, will be furnished and laid by the Railroad Company.

Conduits and Minimum Size of Wire.

251.—If wires are to be placed in conduits, the conduits shall be f ample size, sherardized, or loricated on the inside. No wire less han No. 12, B. & S. gauge, shall be used.

Condulets and Factory Ells.

252.—Conduits shall be of sufficient size to allow the wires to be asily drawn in. No joints are to be made inside of a conduit. Condu-ts and factory ells shall be used. Condulets, ells, and conduits shall e sherardized, or loricated inside.

Specifications for Special Metals Used for Machinery Parts.

Steel Castings.

Qualities of Steel Castings.

253.—Steel for castings may be made by the open-hearth or crucible rocess.

254.—All castings shall be annealed, unless otherwise specified.

Phosphorus	0.05% maximum.
Sulphur	0.05% maximum.

255.—Minimum physical qualities, as determined on a standard est specimen of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch	70 000
Elongation: percentage in 2 in	18
Contraction of area: percentage	25

256.—A test to destruction may be substituted for the tensile test, n the case of small or unimportant castings, by selecting three cast-ngs from a lot. This test shall show the material to be ductile, ree from injurious defects, and suitable for the purpose intended. lot shall consist of all castings from the same melt or blow, annealed the same furnace charge.

Flaws in Castings.

257.—Castings shall be true to pattern, and free from blemishes, aws, or shrinkage cracks. When the bearing surface of any steel sting is finished, there shall be no visible blow-holes exceeding 1 in. any direction, nor exceeding ½ sq. in. in area. The length of blow-oles cut by any straight line laid in any direction shall never exceed in. in any 1 ft.

Blow-Holes in Gear Wheels.

258.—No blow-hole exceeding one-half the above dimensions and rea will be allowed in any gear tooth, or in the rim at the root of the eth.

Electric Welding.

259.—The correction of defects in castings, by welding electrically y thermit or by similar processes, will not be allowed.

Testing of Large Castings.

260.—Large castings shall be suspended and hammered all over. o cracks, flaws, defects, or weakness shall appear after such reatment.

261.—A specimen (1 in. by ½ in.) shall bend, cold, around a diameter of 1 in., through an angle of 90°, without fracture on the outside of the bent portion.

262.—The number of standard test specimens shall depend on the character and importance of the casting. A test piece shall be cut, cold, from a coupon to be moulded and cast on some portion of one or more castings from each melt or blow, or from the sink-heads (in case heads of sufficient size are used). The coupon or sink-head must receive the same treatment as the casting or castings, before the specimen is cut out, and before the coupon or sink-head is removed from the casting.

263.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether or not the steel is within the limits in phosphorus and sulphur specified in Paragraph 254 concerning chemical properties.

Steel Forgings.

Qualities of Steel Forgings.

264.—Steel forgings may be made by the open-hearth or crucible process:

Phosphorus	0.04% maximum.
Sulphur	0.05% maximum.

265.—Minimum physical properties as determined on a standard turned test specimen of ½ in. diameter and 2 in. gauged length.

266.—Tensile strength, in pounds per square inch, 85 000 to 65 000

Elongation: percentage in 2 in. 28

A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of ½ in., without fracture on the outside of the bent portion. The bending may be effected by pressure or by blows.

267.—The number and location of the test specimens to be taken from a melt, blow, or forging shall depend on their character and importance, and, therefore, must be regulated by individual cases. The test specimens shall be cut, cold, from the forging, or full-sized prolongation of the same, parallel to the axis of the forging and half way between the center and the outside; the specimens shall be longitudinal, *i. e.*, the length of the specimen shall correspond with the direction in which the metal is most drawn out or worked. When forgings have large ends or collars, the test specimens shall be taken from a prolongation of the same diameter or section as that of the forging back of the large end or collar. In the case of hollow shafting, either forged or bored, the specimen shall be taken within the finished section prolonged, half way between the inner and outer surfaces of the wall of the forging.

268.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred

by the inspector, shall be used to determine whether or not the steel is within the limit in chemical composition.

269.—Forgings shall be free from cracks, flaws, seams, or other injurious imperfections, and shall conform to the dimensions shown on the drawings furnished by the purchaser, and shall be made and finished in a workmanlike manner.

270.—All forgings shall be annealed.

Machinery Steel.

Qualities of Machinery Steel.

271.—Machinery steel shall be made by the open-hearth or crucible process.

Phosphorus.....0.05% maximum.
Sulphur.....0.05% maximum.

272.—Minimum physical properties, as determined on a standard turned test specimen of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch.....80 000
Elongation: percentage in 2 in..... 20

273.—A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of 1½ in., without fracture on the outside of the bent portion. The bending tests may be made by pressure or by blows.

274.—Turnings from the tensile test specimens, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether the melt is within the limits in chemical composition.

Boiler Plates.

Qualities of Boiler Plate Steel.

275.—The steel used for boilers and fire-boxes shall be made by the open-hearth process.

Phosphorus.....0.04% maximum.
Sulphur.....0.04% maximum.

276.—The physical properties required shall be as follows:

Tensile strength desired, in pounds per square inch...60 000.
Elongation: minimum percentage in 8 in. $= \frac{1\ 500\ 000}{\text{Ultimate strength.}}$
Character of fracture.....Silky.
Cold bends, without fracture.....180° flat.

277.—The ultimate strength shall come within 4 000 lb. of that desired.

278.—Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese, shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of each analysis shall be furnished to the Engineer or his inspector. A check analysis shall be made from the finished material, if called for by the purchaser, in which case an excess of 25% above required limits will be allowed.

279.—Specimens for tensile and bending tests for plates shall be made by cutting coupons from the finished product, which shall have both faces rolled, and both edges milled to the usual form of the standard test specimen, 1½ in. wide on a gauged length of at least 9 in.; or with both edges parallel.

Nickel Steel for Machine Parts.

Qualities of Nickel Steel.

280.—Nickel steel shall be made by the open-hearth process.

	Plates, shapes, and bars.	Rivets.
Phosphorus shall not exceed	0.04%	0.04%
Sulphur	0.05%	0.04%
Nickel, not less than	3.00%	3.25%

281.—The physical properties required shall be as follows:

	Plates, shapes, bars, and forgings. Pounds per square inch. Minimum.	Rivets.
Tensile strength	80 000	60 000 to 70 000
Elastic limit	50 000	40 000 minimum.

Elongations: percentage in 8 in., for plates, shapes, bars, and forgings; and also for rivets $= \frac{1\ 600\ 000}{\text{Ultimate strength.}} =$ minimum.

Elongation: percentage in 2 in. for forgings = 25.

282.—Specimens cut from forgings (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of 1 in., without fracture on the outside of the bent portion.

283.—Specimens cut from plates, shapes, and bars shall bend, cold, 180°, around a diameter of three times their thickness, without fracture on the outside of the bent portion.

284.—Each rivet rod shall bend 180°, flat, on itself, without fracture on the outside of the bent portion.

285.—Rivet rods shall be tested as rolled.

286.—The fracture of all tension tests shall show a fine silky texture, of a uniform bluish gray or dove color, free from black or brilliant specks, and shall show no sign of crystallization.

287.—All nickel-steel forgings shall be properly annealed.

288.—Annealed eye-bars and similar members, when full-sized pieces are tested, shall comply with the following requirements:

Minimum ultimate tensile strength, in pounds per square inch	75 000
Minimum elastic limit, in pounds per square inch	45 000
Minimum elongation in 10 ft., including fracture	12%
The fracture shall be mostly silky, and free from crystals.	
Full-sized pieces shall bend, cold, 180°, around a diameter of twice their thickness, without fracture.	

Tool Steel.

Qualities of Tool Steel.

289.—Tool steel is generally used for parts which require hardening or oil tempering, such as pivots, friction rollers, ball-bearings, and springs.

290.—Tool steel shall be made by the open-hearth or crucible process.

Carbon	1.00% minimum.
Phosphorus	0.04% maximum.
Sulphur	0.04% maximum.
Manganese	0.50% maximum.

Phosphor-Bronze.

Qualities of Phosphor-Bronze.

291.—Special phosphor-bronze shall be used for high pressure and slow speed.

292.—Phosphor-bronze shall be a copper-tin alloy. The phosphorus shall not exceed 1 per cent. Other alloys, up to one-half of 1%, will be permitted, except that no sulphur will be allowed.

Compression:

Elastic limit, in pounds per square inch.	19 000 to 23 000
Permanent set, under 100 000 lb., in inches	0.12 to 0.16

293.—The compression is to be made on a cylinder having a height of 1 in. and an area of 1 sq. in. The elastic limit is to be the load which gives a permanent set of 0.001 in.

Tension:

The yield point, ultimate strength, and elongation in 2 in. are to be recorded. The tension specimen is to have a diameter of $\frac{1}{2}$ in.

294.—For every heat, at least two tests shall be made. A chemical anaylsis shall be furnished.

Babbitt Metal.

Qualities of Babbitt Metal.

295.—Babbitt metal composed of the following ingredients and of the following proportions has given satisfactory results and a low coefficient of friction (0.03 to 0.04):

Copper	3.6 per cent.
Tin	89.3 " "
Antimony	7.1 " "

Purpose of the Specifications.

296.—It is the purpose of these specifications to provide a first-class structure. They are intended as an aid in designing and fabrication. Machine design and kindred subjects are so great and varied that no single work of this character can cover all points. As a further aid in

securing a first-class structure, the following works will be considered authoritative, in the order named:

1. Unwin's "Machine Design," Part I, Ed. 1909.
   Unwin's "Machine Design," Part II, Ed. 1902.
2. "A Manual of Machine Design," etc., by Low and Bevis, 11th Impression.
3. Reuleaux's "Constructor," Translated by Suplee.
4. Kent's "Pocket Book," 8th Ed.

297.—Machine parts shall be designed, if practicable by the methods of applied mechanics, but such designs shall be viewed in the light of experience. It should be borne in mind that machine design is not based on the precise methods in vogue for stationary structures.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1252

A BRIEF DESCRIPTION OF A MODERN STREET RAILWAY TRACK CONSTRUCTION.*

By A. C. Polk, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. E. E. R. Tratman, Walter C. Howe, Louis A. Mitchell, William J. Boucher, C. B. Vorce, and A. C. Polk.

With the increasing demands made on various traction companies hroughout the country to provide good service, the matter of securing permanent roadbed in the busy and paved sections of a city has ecome a serious problem. When the work is poorly done originally, nd is not on a proper foundation, the cost of maintenance is in-reased, the paving along the poor roadbed becomes cracked and uined by the movement of the track, unfavorable public comment on arises, and the company finds itself face to face with one of two ropositions, either the entire rebuilding of the roadbed or the attempt o surface and repair the old one. Either alternative is expensive, as he work will have to be done on a busy street and will interfere with ar schedules, with consequent loss of revenue to the company and nconvenience to the public. Therefore, when the paving is done riginally, the permanent roadbed should be substantial and effective.

In this paper the writer has attempted to give a general descrip-ion of the methods and materials used at Springfield, Mo., in rebuild-ng the Springfield Traction Company's lines on streets which were eing paved by the City during 1911.

* Presented at the meeting of October 16th, 1912.

The Company's franchise stipulates that it shall pave the space between its rails and for 2 ft. outside of the rails, when the City paves on the various streets occupied.

The work was done by the Company, under the writer's supervision, and aggregated more than 3 miles of new tracks.

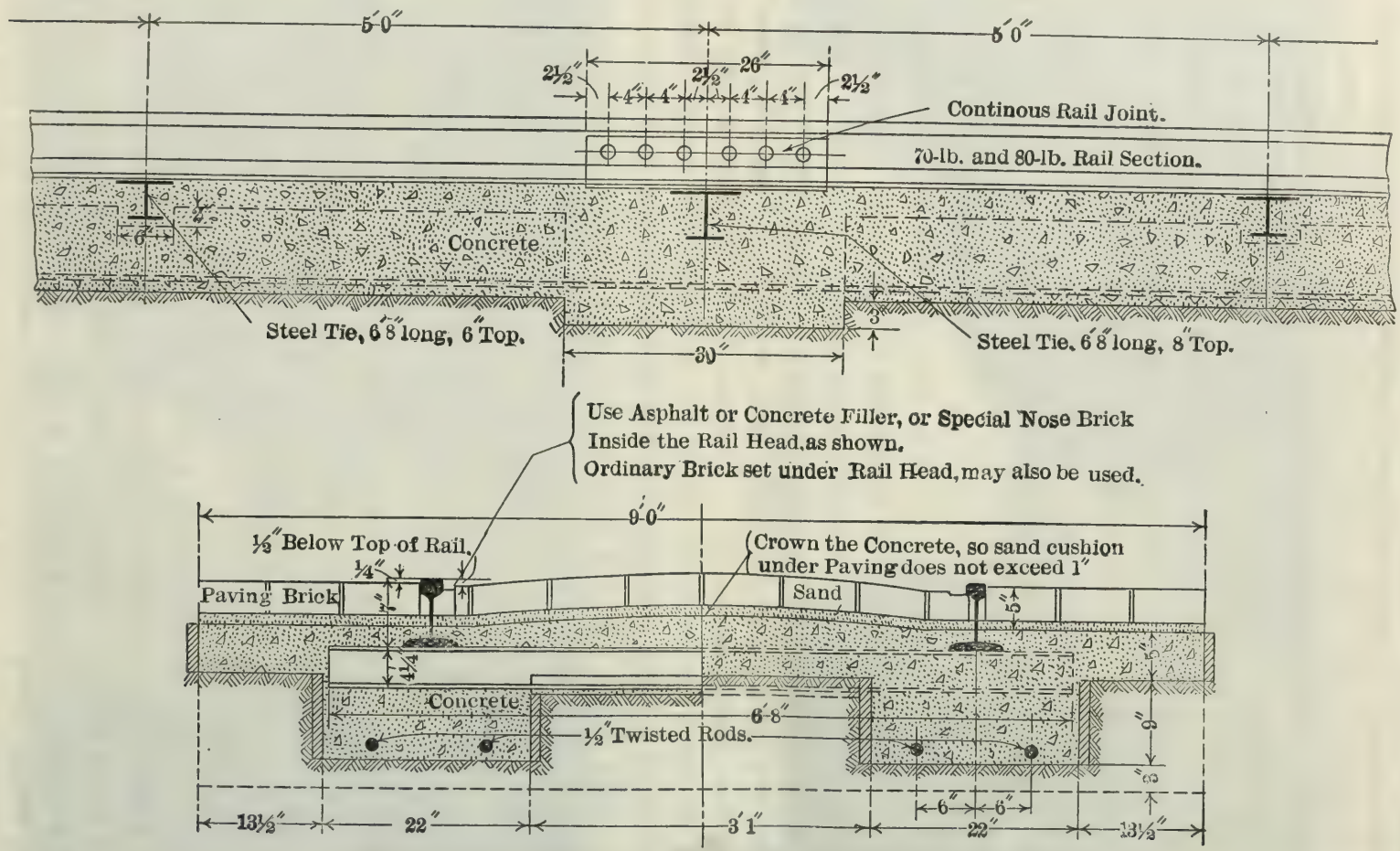
It was desired to secure as permanent a roadbed as possible, in fact, one which would outwear the rail itself and allow its replacement without disturbing the roadbed if so desired. Fig. 1 shows the general cross-section and detail of the work as put in.

General Features.—The general features of the structure were, a 7-in., 70-lb. T-rail section, 62 ft. long, laid on steel ties, and, directly under each rail, longitudinal concrete beams, each reinforced with two ½-in. twisted steel rods, and a 5-in. concrete mat over the beams and the center of the tracks for a paving foundation.

In this case the city contractors were allowed to place the street pavement first, except where it was of asphalt. After the paving on each side of the track had been completed, the old track was moved to one side or the other on the completed pavement, and excavation between the tracks was commenced, as shown on Figs. 2 and 3.

Excavation.—The standard depth below the finished pavement to which the general excavation was made over the whole 9-ft. space was 10 in., as shown by Fig. 1. Then two trenches were excavated to exact dimensions, a depth of 9 in. more, and running with the rail. At the joints these trenches were connected by cross trenches, 30 in. wide, at an additional depth of 3 in., so that each joint tie would be thoroughly embedded and have a good thickness of concrete under it. Around each intermediate tie shallow trenches were excavated so that there would be at least 2 in. of concrete under the tie and 4 in. around its sides.

Ties.—All ties were punched at the proper gauge to take 7-in., 70-lb. or 80-lb. T-rails, but the 70-lb. rail was used principally. There were two types of ties, one under the joints which came opposite one another, and the other intermediate between them. The joint tie was an I-beam section, weighing 20 lb. per ft., 6 ft. 8 in. long, having an 8½-in. top, on which the joint rested, and a 4½-in. base. The intermediate ties were of the same length, but weighed 14.5 lb. per ft. and had a 6-in. top and a 4-in. base. All ties were spaced 5 ft. apart, or as near that as the distance between joints would permit.



SECTION CONCRETE STEEL TIE TRACK CONSTRUCTION.

Fastenings.—The rail was fastened to the tie by special lugs and T-headed bolts with square shoulders next to the head. These bolts could be inserted from the top, if so desired, being dropped through the rectangular hole punched in the tie, turned, and then raised until the square shoulder fitted up into the rectangular hole, which prevented it from turning while running down the nuts. The lug had a projection on its under side which also fitted into the rectangular hole and prevented it from turning or backing off the rail. A socket wrench was used to run down these bolts rapidly. The joint ties were heavier and wider than the intermediate ones, as noted above, and were punched to allow the lug to fasten over the continuous joint which was used.

Joints.—Six-bolt continuous joints were used, and the following method of making them was adhered to with greatest care, as the joint is generally the weak part of the track work:

First, those parts of the rail with which the joint came in contact were carefully cleaned and polished with files and emery cloth until all scale, rust, and particles of dirt were removed and the surface was bright. Then those parts of the joints which came in contact with the rail received the same treatment. All contact parts were then greased and the joint put on. The tightening of the bolts was started from the center, working out toward the ends, pulling up a bolt on one side of the center and then the corresponding bolt on the other side, and when the bolts were all apparently tight a heavy sledge was held against the bottom of the joint on one side and the other side tapped briskly with a light sledge at the bottom only, and *vice versa.*

Then the tightening process was repeated until nothing more could be gotten out of the bolts by one man with a wrench having a 30-in. handle. After a continuous joint has once been carefully made in this manner it is there to stay, and even if the bolts were removed the joint would remain under traffic for a long period without any signs of loosening. Another feature in making a joint of this type in this manner is that no bonds were used, or considered necessary. The net area in contact has a far greater current-carrying capacity than the No. 0000 copper bond generally used on heavy lines, and a test showed no trouble with return circuits. It has also been brought to the writer's attention that a section of track put down in a similar manner showed a first-class negative return circuit after two years under traffic.

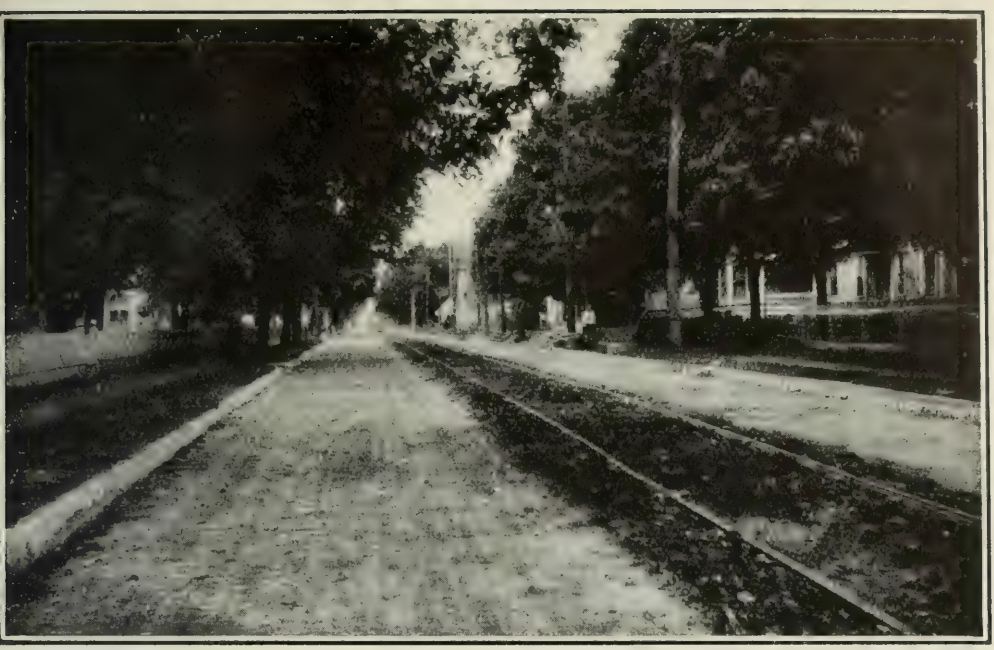


FIG. 2.—STREET AS LEFT BY CITY PAVING CONTRACTOR READY FOR STARTING WORK ON CAR TRACK.

FIG. 3.—OLD TRACK THROWN OVER ON NEW PAVEMENT, EXCAVATION STARTED.

Surfacing and Preparing for Concrete.—The track, having been put together as described, was supported on small piers of old brick placed under every other tie, and centering under the rail. The final surfacing was done with oak wedges on top of these piers, and both the piers and wedges were concreted in.

Two ½-in. twisted steel rods were placed in the bottoms of the trenches about 6 in. apart. These also were supported at intervals on top of a brick which held them about 2½ in. off the ground. Under each joint two extra 4-ft. lengths of these steel rods were placed, to give additional strength at this point, which is usually considered the weak spot of track construction. This is shown on Fig. 4.

The track was then given its first surfacing raise, being brought approximately to grade. Afterward, a second finishing surface and true grade was secured by driving up the oak wedges. This final surface, however, was only kept a short distance ahead of the mixer on account of the difficulty in keeping the skeleton track in correct line, variations in temperature kinking it.

Concrete.—Concreting was next started, two types of mixers being used on this particular work at different times: One was mounted on car trucks and driven with a 10-h.p. motor; it occupied the temporary old tracks which had been previously placed at one side, and was supplied by a regular work train which delivered the concrete material on flat cars. The other type was mounted on wide-tired wheels, driven with a 15-h.p., 500-volt, D. C. motor, and was equipped with a charging bucket. All materials for the latter were hauled in wagons to the streets and dumped ahead of the mixer in such manner that they were about in the right proportions. These materials were cleaned up as the mixer progressed.

Of the two methods, the latter was found to be by far the more satisfactory, as there were frequent delays in the delivery of materials to the first mixer by the work train, and the temporary track was blocked for traffic beyond the point where concreting was in progress and, on single-track work, passengers had to transfer around the obstruction.

In the second method, all material could be delivered on the street for a considerable distance ahead of the mixer, and there was no reason for the mixer to be idle at any time on this account.

The concrete was composed of 1½-in. (maximum size) crushed limestone, Kaw River sand, and chats from the Aurora Zinc Mine district, the proportions being one of cement, two of sand, two of chats, and three of crushed stone, this making a very good mixture. The concrete was mixed very wet, dumped into movable chutes from the mixer, and deposited directly by them on the track. Great care was taken at all times to keep all loose material in the excavation cleaned up ahead of the concrete and to maintain the twisted steel rods in their proper positions. The concrete was puddled carefully under and around all ties with shovels, and finally crowned, when it reached grade, with a special board or templet made for the purpose.

When the material through which excavation had been made was of a very loose character, or if bad weather had caved in the sides of the trench, forms were placed where necessary to hold the concrete to the exact dimensions of the roadbed. After the concrete had set for 48 hours, and in the majority of cases longer, the brick paving was done.

Paving.—The paving was of first-class paving blocks, laid on a 1-in. sand cushion, and grouted. The method of laying it differed somewhat from the usual practice. Along each rail, and snug up against it, was laid a 2 by 4-in. piece of timber. A stretcher course of bricks was then laid against it paralleling the rail, the tops of the bricks being only ½ in. below the top of the rail. The headers across the tracks were then laid and all the bricks were thoroughly rolled and rammed.

Before grouting, the 2 by 4-in. piece was removed and the space thus left was filled with concrete to a plane 1 in. below the top of the rail, forming the flangeway for the wheels. After this had set, the whole pavement was grouted thoroughly and carefully. When filled up, the grout remaining over the top of the pavement was allowed to stand until it had attained a set and was fairly stiff, then dry sand was sprinkled over it and the whole was swept thoroughly with wire brooms, thus removing all surplus grout sticking to the tops of the bricks and presenting a very neat, clean surface. It was noticeable, too, how very much the paving contractor improved in the character of his work, when this specification of cleaning off the top of the brickwork was strictly enforced. By this method all irregularities

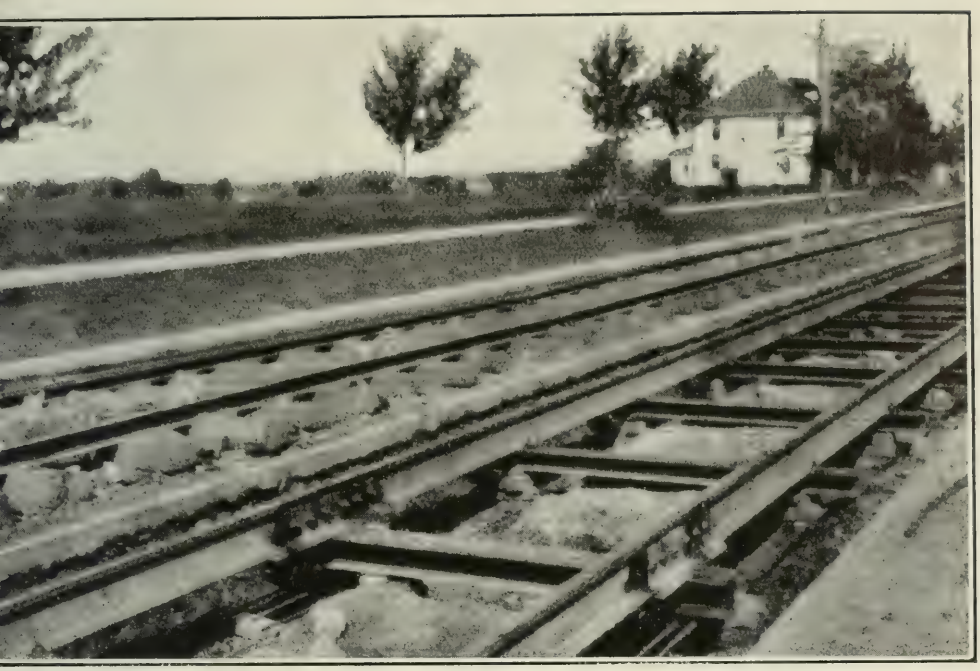


FIG. 4.—REINFORCEMENT AT JOINTS, AND GENERAL CONSTRUCTION.

FIG. 5.—CONCRETE AND TRACK WORK COMPLETE, READY FOR BRICK PAVING.

of laying and surfacing are strongly brought out, and easily seen and corrected.

Cost.—The cost of track construction of this type for paved streets, including the paving between and for 2 ft. outside of the rails, using the 70-lb. high T-rail, under the conditions at Springfield, was approximately $5 per ft. of single track. This cost may seem to be high, but, when the permanent character of the work is taken into consideration, and the cost of doing street work under other methods is analyzed and compared, it will be found to be very reasonable. It possesses several advantages over some of the methods in vogue. It takes less concrete than the old method of filling under and around wooden ties. It places the concrete and the strength where it is most needed, and, when properly put in, it is certainly permanent, and the roadbed will easily outwear two or more sets of rails under heavy traffic. The rail can be changed without disturbing the roadbed in the least, if proper provisions have been made in the steel ties originally.

On one section of similar track construction, known to the writer, not the slightest sign of any motion could be detected after 3 years' operation, and it is reported to be in as good condition as the day it was built.

The writer trusts that the foregoing brief description may prove of interest to members of this Society and bring out some discussion on work of this class, concerning which very little has appeared in the technical press.

The work was done by the Springfield Traction Company, owned and operated by the Federal Light and Traction Company, of New York, and was executed under the general direction of Mr. W. A. Haller, Chief Engineer, the writer being in immediate and full charge of all construction matters at Springfield, Mo.

DISCUSSION

Mr. Tratman.

E. E. R. TRATMAN, ASSOC. M. AM. SOC. E. (by letter).—An interesting point suggested by this paper is the shape of the flangeway groove along the rail. It appears to be a rather wide, rectangular groove, having a flat bottom, vertical face, and sharp corner. It will be of interest to know whether this vertical face stands up under traffic, or whether the wheels of vehicles crush and crack it to an approximately beveled outline. The more usual method of forming the groove is to use nose-brick laid as headers with the top surface next the rail beveled so as to fit beneath the rail head. Still another plan is to use ordinary bricks (also laid as headers), tilted so that one end fits under the rail head while the top surface lies approximately in the contour of the crowning of the pavement between the rails. A third plan is to use a rectangular stretcher under the rail head (and projecting beyond it) and a higher bevel-edged stretcher level with the paving. In all these methods the groove or flangeway is of beveled or triangular section rather than rectangular.

In the Springfield work, the crowning of the paving between the rails, as shown in Fig. 1, appears to be unduly high, interfering with the normal cross-section of the street, and making an irregular contour. This is especially the case as the paving is $\frac{1}{2}$ in. below the rail head on the inside and $\frac{1}{4}$ in. below it on the outside, so that each rail forms a distinct ridge in the pavement. This construction does not seem to be desirable in connection with a well-paved street surface, and it would be of interest to know whether the city authorities or the public have made any objection to it.

In regard to the wide spacing of the ties, experience has shown that the concrete immediately beneath the rail disintegrates or crushes, in some cases, due to the vibration and slight deflection of the rail, in spite of the fact that, theoretically, the rail has a continuous solid bearing.

Mr. Howe.

WALTER C. HOWE, ASSOC. M. AM. SOC. C. E. (by letter).—This paper is both interesting and instructive. The writer has found very few published data on work of this class, although it is one of the most important features in the construction of modern pavements. Municipal engineers are often at a disadvantage in drawing up ordinances and specifications governing the control of paving in the excepted portion occupied by street railways.

The writer was Commissioner of Streets of Oakland, Cal., for a number of years, and observed that the initial point of failure of all asphalt streets was that portion immediately contiguous to the rails of the street railways, of both steam and electric roads. The asphalt surface adjoining the tracks was in a continual state of disrepair and

FIG. 6.—ASPHALT PAVING, TWENTY-SECOND STREET, OAKLAND, CAL. GIRDER-RAILS, STEEL TIE-PLATES, WOODEN TIES, CONCRETE TO BASE OF TIE, BASALT BLOCK LINERS ON 1½-IN. SAND CUSHION.

FIG. 7.—ASPHALT PAVING, FOURTH AVENUE, OAKLAND, CAL. VITRIFIED BRICK LINERS. ASPHALTIC CONCRETE BETWEEN TIES, INSTEAD OF HYDRAULIC CONCRETE.

sintegration. The failure of the pavement gradually extended out- Mr. Howe.
de of the railroad's portion and into that of the city. These conditions
adually became so serious that it was found necessary to adopt new
nd drastic ordinances governing the type of construction to be done
steam and traction lines occupying city streets. Few, if any, rails
modern type had been used by the companies previous to this action.
type of construction somewhat similar to that described by Mr. Polk
as considered. The concrete extended to a depth of 6 in. under and
mpletely across the length of the ties, but no steel reinforcement
as used in the foundation. Strong opposition to this type of con-
ruction was made by the railway companies, and, as a compromise,
ck ballast 6 in. deep was substituted for the 6 in. of concrete under
e ties. There is no doubt that the class of construction shown by
r. Polk is far superior, embodying, as it does, steel ties and a solid
einforced concrete foundation under them, thus affording extreme
gidity with a naturally decreased vibration. Its cost, however, is
ch as to preclude its use except in the case of street railway com-
anies who are in excellent financial condition and whose officials are
sposed and ready to meet with the municipality and adopt the most
odern type of rail and foundation.

In many instances the official in charge of paving construction
a municipality is blamed for the dilapidated condition of paving,
oth adjoining and between the tracks of street railways, when, as
matter of fact, the entire trouble is due to the lack of co-operation
n the part of the traction officials, or poor and worthless ordinances
overning the control of the work, coupled with lack of support from
e city fathers.

The writer makes no criticism of the construction described by
r. Polk, but in the case of those who are about to go through the
nenviable struggle of forcing public service corporations to use suit-
ble construction to obviate ruined and dilapidated pavements adjoin-
g railroad tracks, it may become necessary to follow an intermediate
ourse, and in such case the construction described herein can be
ut down with good results. This type is not new by any means, but it
mbodies several unique features, such as a modern, 141-lb., 9-in.
irder rail, with steel tie-plates and light concrete foundation; the
bstitution of asphaltic concrete for hydraulic concrete in some cases;
e laying of paving blocks on their broad flat faces instead of on
dge, etc., etc.

Previous to 1907, 90% of the asphalt and bituminous pavements
ntiguous to street railways in Oakland were in the condition shown
Fig. 6. From time to time, efforts had been made by the trac-
on lines to tooth the rails with basalt blocks in order to avoid this
isintegration. No results were secured, as the asphalt surface broke
p outside the line of the blocks. The condition was due simply

Mr. Howe.

to excessive vibration caused by the light T-rail used, coupled with lack of foundation and inferior roadbed. Under the existing State laws and the ordinances of the municipality, railway companies were required to pave their portion of the street, and for a distance 2 ft. outside of the outer rail, with the same class of pavement as that on the remainder of the street. Specifications governing new asphalt streets called for a concrete base of 6 in., a binder course 2 in. deep, and an asphalt wearing surface of 1½ in., making a total depth of 9½ in. for the completed pavement. Most of the rails laid throughout the city were the old type of light T-rail, ranging between 5 and 7 in. in depth. This gave approximately 2½ to 4½ in. of concrete below the top of the tie. The question as to whether the company could be compelled to increase the depth of the concrete between the ties and also line the rails with blocks, when the State laws required "that paving should be similar in all respects to the remainder of the street" was a legal one, the traction companies taking the stand that they were not re-

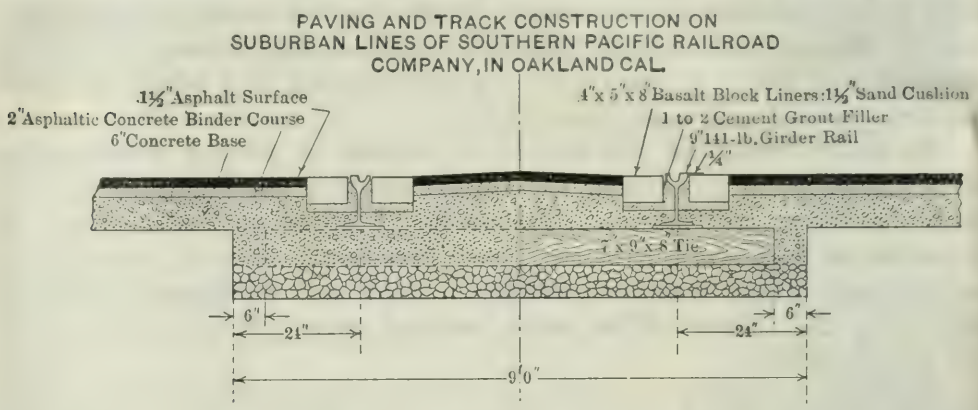


FIG. 8.

quired to do more than was called for by the State law. After many conferences and considerable discussion on the part of the city and traction officials, ordinances were drafted requiring the companies to line their tracks with types of paving brick or stone blocks satisfactory to the city, and to lay concrete the full depth of the ties. This type of construction was subjected to considerable criticism at the time. It was argued that the rigidity desired could not be secured unless the foundation concrete was carried to a depth of at least 6 in. under the ties. The ordinance was adopted despite the criticism, and work was done on many miles of streets. The results have been uniformly satisfactory, notwithstanding the apparently cheap construction.

Fig. 7 shows the west half of a completed asphalt street with basalt block liners toothing the rails. The concrete foundation on the right is complete and ready to receive the binder and surface materials. The open space adjoining the rails is ready to receive the block liners. The concrete was carried the full depth of the

FIG. 9.—CONDITION OF ASPHALT PAVEMENTS ADJOINING CAR TRACKS WHERE NO LINERS ARE USED. OLD TYPE T-RAIL, AND LIGHT CONCRETE FOUNDATION BETWEEN TRACKS.

FIG. 10.—ASPHALT PAVING, BROADWAY, OAKLAND, CAL. BASALT BLOCK LINERS TOOTHING THE RAILS.

ties, practically encasing the wooden tie in a concrete body. Many of the ties showed signs of dry rot; the worst were removed, but many not badly affected were left in place. Owing to the depth of the rail, the basalt blocks were laid flat, instead of on edge, which considerably lessened the number used and naturally the cost. The blocks were laid on a sand cushion about 1 in. deep, and a 1:2 cement grout was swept into the joints. During the process of concreting, the traffic was carried on one track only, cross-overs being built at intermediate points. This allowed the concrete to set thoroughly before being subjected to the strain of traffic operations. This type of construction has proven very successful, although on a street carrying the cars of four branch lines controlled by the same company. Mr. Howe.

Fig. 9 shows the completed asphalt pavement up to the line of the Key Route Company's tracks. On this street 141-lb. girder rails, 9 in. deep, were laid, with tie-plates as shown. The concrete extended only to the full depth of the tie. Spaces on both sides of the track are shown ready to receive the sand cushion and basalt blocks before laying the binder course and asphalt surfacing. Heavy interurban trains operate over this line.

Fig. 8 shows the type of construction on the Southern Pacific interurban lines. Girder rails of the same type were used, with basalt blocks set on edge. These blocks ranged in depth from 5 to 6 in.

Fig. 10 shows the type of construction used on Fourth Avenue, Oakland. Vitrified brick liners were used in place of basalt blocks, and the space between the ties and to their full depth was filled with a rich asphaltic concrete instead of hydraulic concrete. The old type of T-rail used is shown. As this line was single-tracked for a considerable distance, it became necessary to maintain traffic during the paving operations. For this reason, it was thought inadvisable to put in hydraulic concrete and operate the cars at the same time. To overcome this, asphaltic concrete was substituted and thoroughly tamped between the ties, being finally rolled with a 2½-ton tandem roller. This type of construction had been used on a number of other streets, but had not proved entirely successful. This work has been completed for several years, and when seen by the writer a few months ago was in very good condition, considering the type of rail used.

Asphalt oil bricks, similar to the Eastern type of asphalt blocks, were used as liners on several streets, but proved to be a failure. These bricks soon pounded into a uniform mass under turnout traffic from teaming, and thereby lost their individuality, taking on the general appearance of the asphalt surface adjoining. They proved to be inferior to the asphalt surface, as disintegration invariably occurred after the heavy rains, continuing during the heavy weather until the remain-

Mr. Howe. ing portion of the block had to be removed. Asphalt blocks of better quality may obviate the earlier failures.

The construction described herein is not recommended for adoption wherever the more modern type can be secured. The best is none too good, as far as paving operations are concerned.

This type of construction has been generally adopted in Oakland, Cal., after a most successful experience covering a period of about 4 years.

Mr. Mitchell. LOUIS A. MITCHELL, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest because similar problems are constantly coming up to be solved.

In the writer's experience, concrete beam construction, of which that described by Mr. Polk is a type, has not been very satisfactory for railroad tracks in paved streets. Its failure, however, has not been due necessarily to the type of construction nor to the area of the cross-section of the beam; though there is no doubt that some failures have been caused by the provision of insufficient bearing, and the fact that the loads have been too great. Most failures, however, have probably been due to loose joints which allow a slight movement of the rails when a wheel passes over them. This movement starts a hammer which the concrete will not stand, and, ultimately, the beam breaks under the joint.

By placing a steel tie directly under, and thus supporting, the joint, Mr. Polk has taken a step in advance in concrete beam construction, which, no doubt, will lengthen the life of the foundation. This support will prevent the working of the ends of the rails and the hammer on the concrete. The writer believes that the life of the joint will be greatly lengthened if the support under it is increased, at least, to the full length of the joint-bar, or longer. He has used steel ties constructed of 4-in. channels, 36 in. from center to center, and connected at the ends with $\frac{3}{16}$-in. steel plates. The rails rested on these steel plates, and the joint was placed in the center so that the rails were supported for a distance of 18 in. back from the joint. Simple joint-bars 26 in. long, with six bolts, were used. These ties were fastened to the rail by castings which fitted over the base of the rail and through the steel plates (which had been previously punched) and under the rail. These castings were held in place on one side of the rail by pieces of $\frac{3}{16}$-in. plate, which were placed back of the casting and extended through the plate. The steel tie and the base of the rail were then concreted in the usual manner. This type of construction has proved very satisfactory for tracks on which 45-ton cars are operated.

The work described by Mr. Polk does not deal with the most important part in the making of a good joint for paved streets, although he may have taken care of this properly. When bolts are

ed, the holes in the web of the rail and in the joint-bars should be the same size as the bolt, and machine-bolts should be used, mak- g a driving fit. The ends of the rails should be ground so that they ll fit tightly, especially at the ball. The drilling should be such at when the bolts are in place the ends of the rails will fit tightly gether. This type of joint allows for no change of length in the il due to change in temperature; but this is not necessary owing to e fact that in paved streets only a small portion of the rail is ex- sed to extreme changes in temperature, and the tendency to change length is taken up in internal stresses. Mr. Mitchell.

Any open joint will permit the wheels of a car to pound, and this ill increase until the ball of the rail is ruined. This pounding may maintained at a minimum, even in open joints, by keeping the rails ound so that they are of the same height at the joint.

Another cause of pounded joints, and one which will produce ilure just as surely as an open or loose joint, is difference in the ight of the rails. Specifications for rails allow for a possible riation in height of $\frac{3}{64}$ in. at any joint, and even if the joint tight in the bolts and the rails well fitted, a pound will start, and ere will be cupped rails in the track in a surprisingly short time. or this reason, the joints of new track should be ground to an even rface just as soon as possible after the work is completed.

In track constructed as described, the writer has seen joints which ere ground soon after the work was completed, and it required a ther close inspection of the ball of the rail even to find where they were.

The writer's work, however, is not of concrete beam construction; some cases there is concrete under the whole track, and in others e track is on broken-stone ballast with concrete from the bottom up 2 in. above the top of the ties. The track constructed by both ese methods and with joints of the type previously described, has quired no maintenance whatever since it was installed $1\frac{1}{2}$ years ago, t it was found that, in some cases, even with the tight joints, the ils had cupped, due to the difference in height, and it was neces- ry to grind the joints to an even surface.

Another method of constructing a tight joint is similar to that scribed, except that rivets are used instead of bolts, thus assuring a ght fit between the rail and splice-bar.

WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—In 1907, the fran- ises of many, practically all, of the street railway lines in Chicago had pired, and, through neglect of maintenance, mismanagement, and fficulties in financial affairs, the tracks and equipment were in a iserable and dilapidated condition. By popular vote, new franchises ere granted, and by the ordinances of February 11th, 1907, the city came a partner in the street railway business, receiving 55% of the t receipts, and the Board of Supervising Engineers was created. The Mr. Boucher.

Mr. Boucher.

duties of this Board, as the name indicates, consisted of complete supervision of all the work necessary (including purchase of materials) to place the roadbed, tracks, cars, car-houses, power-houses, and sub-stations in first-class working condition.

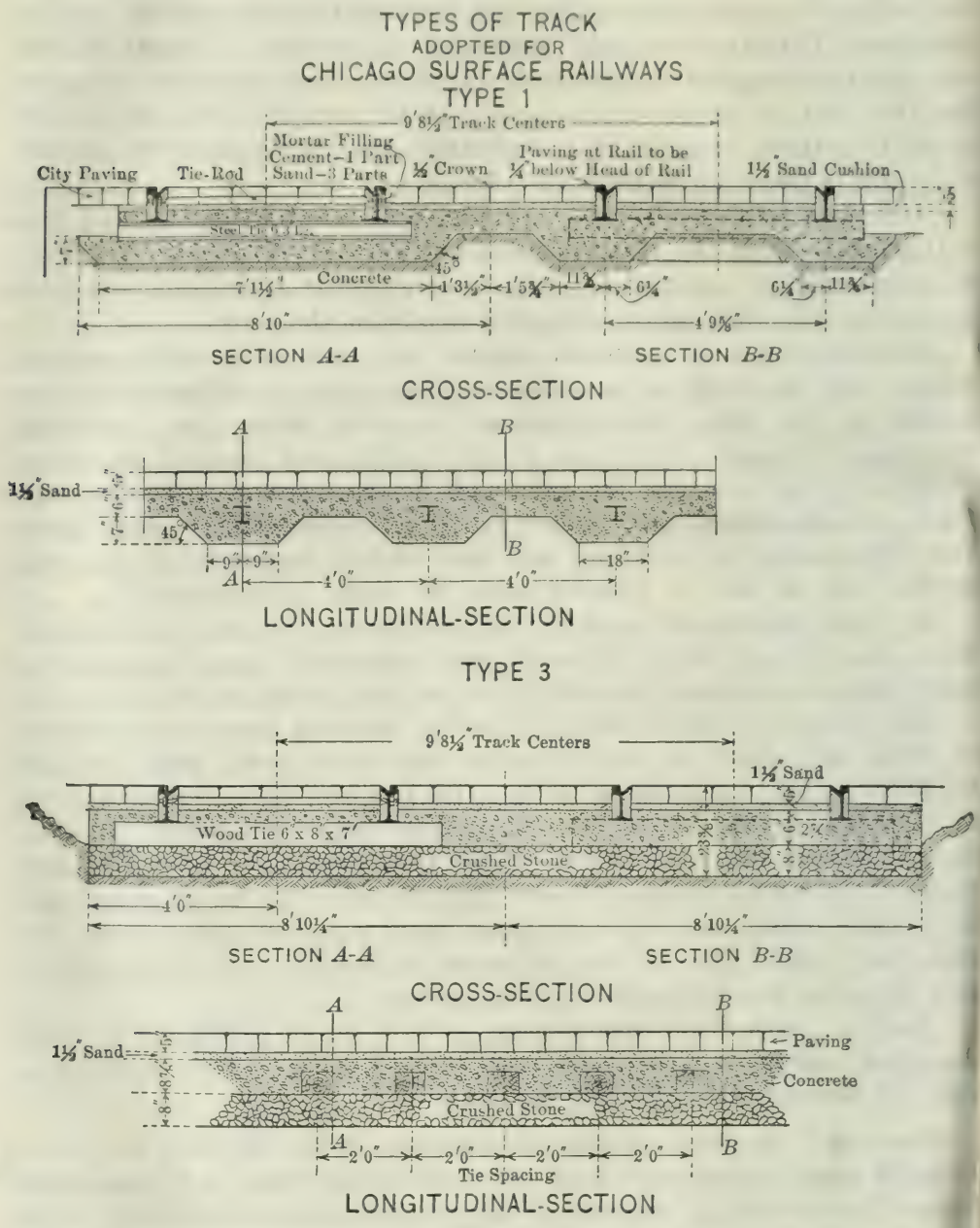


FIG. 11.

The track work was completely remodeled. The first design, known as Type 1 (Fig. 11) was very similar to that described by Mr. Polk. It consisted of steel beam ties, embedded in concrete, which was of greater thickness beneath the ties and rails than between them, where

Mr. Boucher.

ıe original earth was left in mounds, undisturbed. The ties were ·iginally placed 4 ft. from center to center. The concrete embedding ıe ties was brought up flush with the surface of the top flange, which as wider than the bottom flange. These ties are 4¼ in. high, 6 ft. 3 in. ng, and weigh 14½ lb. per ft. The tie is used merely to hold the ils down and to line and gauge; it is not depended on to transmit ıe car load to the soil, that function being performed by the con·ete immediately in contact with the rail. A special form of fastening as designed in order to permit the removal of the rail for renewals ithout disturbing the tie. After the rail was fastened in place on ıe tie, mortar filling was placed over its flange and upward on each de of the web, and packed under the head of the rail. Tie-rods, with vo nuts at each end, pass through the webs of the rails. These rods ·e 2 by $\frac{5}{16}$ in. in section, which permits paving, of granite or wood ocks, between the tracks, the rods being placed in the usual spaces etween the blocks.

Type 2 was a modification of the preceding, and differed from it ainly in the substitution of 6 by 8-in. oak or yellow pine ties, 7 ft. ng. Tie-plates are placed under the rails, and the latter are secured the ties by screw-spikes.

Type 2-A was designed from Type 2, and in it the ties are of wood. hey are 3 ft. from center to center, and the excavation has been hanged to a uniform section, replacing the mounds of earth between ıe ties with concrete and permitting the rolling of the excavated ·ench before placing the concrete. This type is now standard for all ·w work in all parts of the city beyond the limits of the "Loop" section.

Type 3 (Fig. 11) is used in the business or "Loop" section, where ar, vehicle, and pedestrian traffic is very heavy and congested, and here the necessary time cannot be given for concrete to set properly, ıd also for the reason that the streets in this section are likely to e torn up frequently for conduit, water pipe, or sewer work or subway onstruction. In this type the sub-grade is rolled and on it is placed ½- or 2-in. broken stone. This stone is well tamped, on it are laid ooden ties, 2 ft. from center to center, and the usual track structure built.

For all types the straight track rail weighs 129 lb. per yd. It is a rder rail 9 in. high, with 6-in. bottom flange and ½-in. web. Guard ıd curve rails are of similar dimensions, but weigh 145 lb. per yd. he chemical composition is as follows:

For the straight rail:

Carbon	0.50 to 0.60	per cent.
Sulphur, not to exceed	0.08	" "
Phosphorus, not to exceed	0.10	" "
Silicon, not to exceed	0.20	" "
Manganese	0.80 to 1.10	" "

Mr. Boucher.

For the guard-rail:

Carbon	0.45 to 0.50	per	cent.
Phosphorous	0.10	"	"
Sulphur, not to exceed	0.08	"	"
Silicon, not to exceed	0.20	"	"
Manganese	0.70 to 1.00	"	"

Wood ties treated with preservatives have been used extensively in the newer types of tracks.

All joints on straight track are electrically welded. This work is done under contract by one of the rail-rolling companies with its own equipment mounted on special cars, which run on the new track, and current is obtained from the trolley wire. The ends of the rails are butted carefully, but not welded. The weld is accomplished by placing against each side of the webs of the abutting rails, a steel bar, about 1 by 3 by 7 in., on each end of which is a boss, about $\frac{1}{8}$ in. high, the object of the latter being to insure good contact against the web. The bars are held rigidly against the web by heavy jaws operated by hydraulic pressure, and when thus held, a flux is applied and electric current of low voltage and exceedingly high amperage is turned on, the weld being completed in from 3 to 4 min. The heads of abutting rails are afterward smoothed down by a carborundum wheel. The conductivity of the joints must be equal to that of the rails joined.

This construction forms the smoothest running track the speaker has ever ridden over on any street surface railway.

Mr. Vorce.

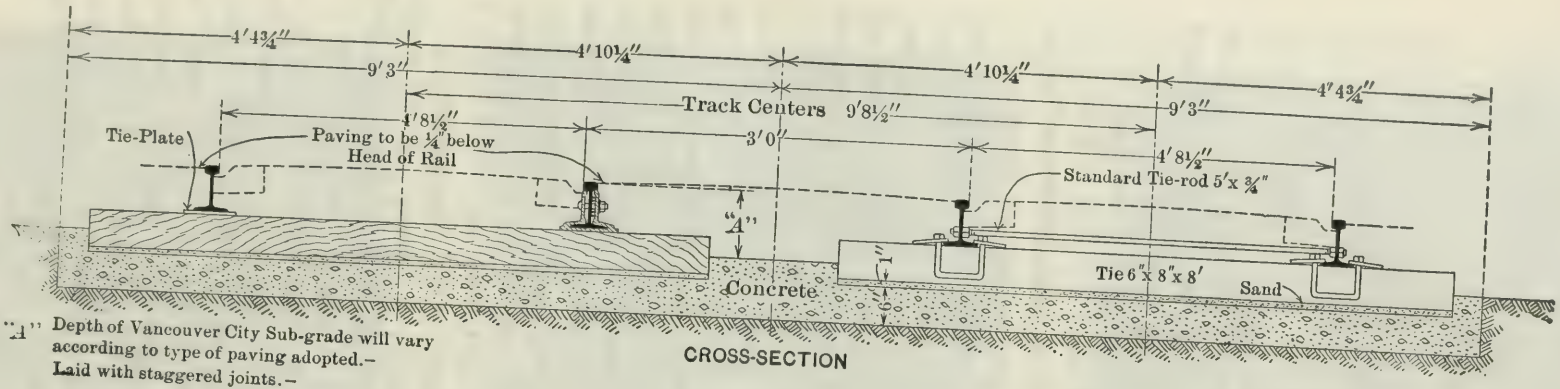
C. B. VORCE, M. AM. SOC. C. E. (by letter).—The writer has read Mr. Polk's paper with much interest, and is glad that a matter of such vital importance to so many members has been brought to the attention of the Society.

As an expenditure of many thousands of dollars is involved, a very careful study should be made before a type of permanent track construction is adopted. In many cases, failure has occurred by reason of a lack of study of the character of the foundations, and a type not at all suitable to existing conditions has been chosen simply because other railroads have used that type successfully.

The writer has had very poor results with the girder type of construction adopted by Mr. Polk. This type was the standard when the writer became connected with the British Columbia Electric Railway Company, Limited, of Vancouver, but, on account of poor foundations, the girders had broken badly, with most disastrous results to both pavement and track.

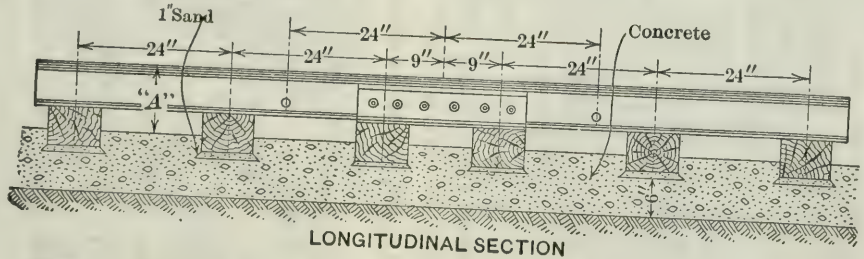
After a careful study, the writer recommended, and the management adopted, the type of construction shown on Fig. 12. Since that time about 15 miles of double track of this type have been laid, and as yet no signs of failure can be seen.

Mr. Vorce.

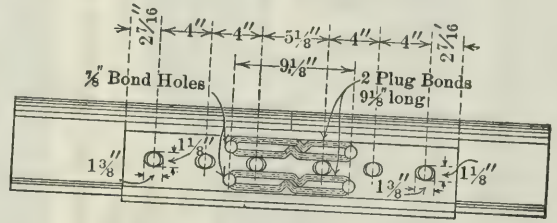


"A" Depth of Vancouver City Sub-grade will vary according to type of paving adopted.—
Laid with staggered joints.—

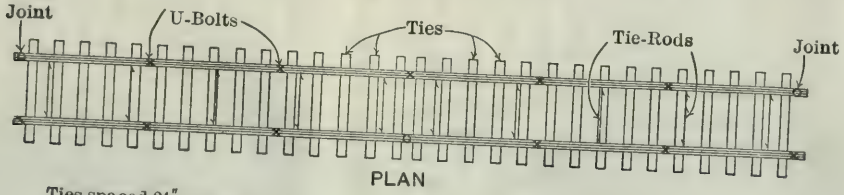
CROSS-SECTION



LONGITUDINAL SECTION

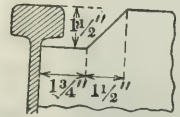


DETAIL OF DRILLING AND BONDING



PLAN

Ties spaced 24″centers.—
Tie-Rods spaced 24″from each joint and the tie-rods between these spaced equally. Total 10 tie-rods per rail length.
U-Bolts spaced 10′apart.—



DETAIL OF FLANGEWAY

TRACK CONSTRUCTION. TYPE NO.1
B.C. ELECTRIC RAILWAY CO., LTD.

Fig. 12

Mr. Vorce. It is the writer's opinion that it is possible to build too rigid a track. He is trying to avoid a hard, rigid, inelastic track, which increases both car and track maintenance, and for this reason he criticizes the steel ties embedded in concrete used by Mr. Polk. The concrete between the ties and under the rail will shrink, allowing the rail to move slightly. The writer is in favor of wooden ties, spaced 2 ft. from center to center, surfaced on about 1 in. of sand, which acts as a cushion. This type has given excellent satisfaction in Vancouver, and is not as noisy and rigid as the steel tie construction.

As to the life of the wooden ties: On special work in Vancouver, ties which have been embedded in concrete for 14 years have been taken up and found to be perfectly sound.

Although Mr. Polk goes into seemingly great refinement to get a tight joint, he does not mention the grinding of the rail ends so that each shall be in the same horizontal plane, which is a matter of vital importance. The rail ends vary, and, no matter how slight this variation may be, unless they are ground to the same height, pounding at the joints will occur, cupping will commence, and the track will fail at the joints while the remainder is in perfectly good condition. When it is remembered that the joints are only about 6% of the total track length, it is readily seen to be good business to make the joints as nearly perfect as possible.

Although the writer's company has bought the best paving bricks obtainable on the Pacific Coast, the results with them in forming a flangeway on the running side of the rail have been very poor, as the bricks break off at the nose very quickly; for this reason the company has been forced to adopt granite flangeway blocks, although the cost is at least 50% greater.

As many of the streets in Vancouver are paved with wood blocks, which are narrower than the granite flangeway blocks, no bond is possible, and a straight joint parallel to the rail results (Fig. 13). With the girder construction, it is almost impossible to keep these flangeway blocks in place, due to the moving of the rail on account of broken girders. Since the adoption of the slab type of construction, which gives a solid foundation, there has been very little trouble.

During the coming season the writer proposes to lay a concrete monolith, 8 in. wide, parallel to the running side of the rail, in which the flangeway is formed. As this will be laid at the same time as the pavement foundation and form a part of it, it is hoped that this will overcome the difficulty of loose flangeway blocks, and reduce the maintenance charges.

In selecting a type of permanent track construction in Vancouver, one must remember that the conditions are not the same as in older established cities where macadamized streets have been in existence for years and a good foundation is available. Owing to the poor founda-

G. 13.—Laying Granite Flangeway Blocks on Granville St., Vancouver, B. C.

G. 14.—Tracks Removed—Grading in Progress—Temporary Track on Left.

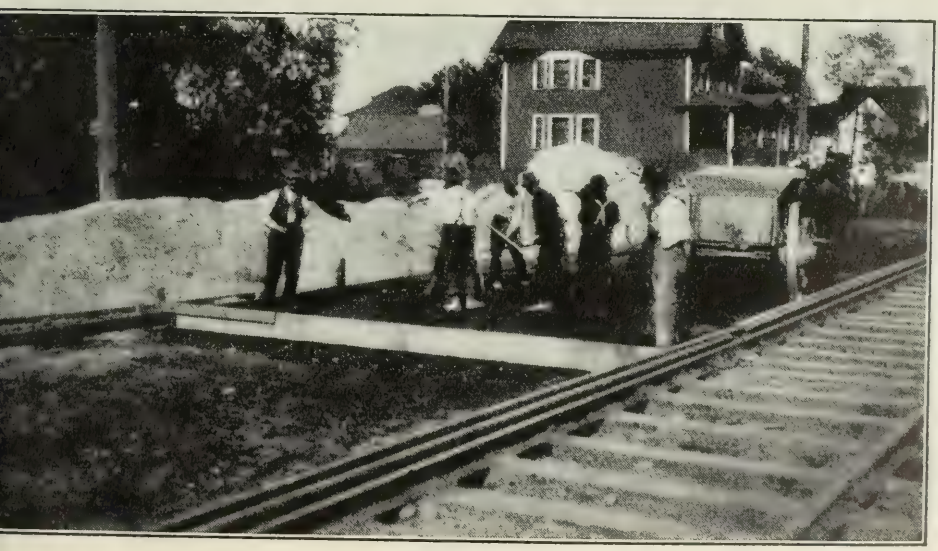


Fig. 15.—Grading Completed. Temporary Track on Right, Broadway, Vancouver, B. C.

Mr. Vorce.

tion, the streets were first planked and then macadamized, the macadam seldom being renewed before laying the permanent pavement on a concrete base. For this reason the foundation is very poor, and the problem has been the selection of a type of construction to meet the existing conditions.

On paved streets, the company always lays a double track, and, as the City pays for all paving for the entire width between the curbs, including foundation, the railway company's liability covers only whatever construction it decides to put in below it. Before the writer took charge of this work, although under the franchise the company could do its own portion of the work, it had been the practice to let it to the paving contractor. Such poor results were obtained that the writer organized and equipped a construction department, and, with few exceptions, the company now does all its own work. At the suggestion of the writer, in order to facilitate the work, the company entered into an arrangement with the City to do all the work in the track allowance, about 18½ ft. wide, the City paying for its share at the same rate as was paid to the contractor for the remainder of the street. This enabled the railway company to do its portion of the work first, the contractor following up and bringing his work to the grade established by the rails. This plan has given most satisfactory results.

The work is carried on in the following manner: A temporary track is laid, outside of the track allowance, to take care of the traffic during construction, thus enabling the company to build both tracks at once. When the traffic is turned over to the temporary tracks, the old track is torn up and the ties and rails are piled on the side of the street, ready to be picked up by the work train, grading is then started (Fig. 14), and the cut is taken out to rough grade; forms are then set by the company's engineer to the proper grade for the top of the slab, and a small gang of fine graders follows up the rough graders; in the meantime the gravel for the slab is delivered along the side of the excavation (Fig. 15). As soon as a block has been rough-graded, work on the concrete slab is commenced, the material being shoveled directly into the hopper, on which a line has been drawn to insure the right proportions. The aggregate is then run through the mixer and delivered into a bucket which travels along a 30-ft. boom cable swinging through 180° (Fig. 16). This method reduces the number of men around the mixer to the minimum, which is an important item, as common labor is paid 37½ cents per hour. All the company's mixers are of this type, and can mix about 80 cu. yd. per 10 hours, or about 250 lin. ft. of completed slab 18½ ft. wide and 6 in. deep. The rails are distributed by an electric derrick car and the ties by the company's motor car. As soon as the slab has set for 24 hours, ties are distributed, T-rails (70-lb., 264, Lorain Section, or 87-lb., 399, Lorain Section) are

Mr. Vorce.

laid, and the track is surfaced to grade with sand (Fig. 17). The standard plan calls for 1 in. of sand under the tie, but, owing to irregularities in the concrete, the thickness varies from ¾ to 1¼ in. After the track has been surfaced, the sand is scraped out from between the ties, the flat wheels on the mixer are replaced by flanged wheels, so that the mixer can run on the track, and the final concrete course is laid (Fig. 18).

It will be seen that the four operations, grading, laying the slab, laying the track, and putting in the final concrete course, are all carried on at the same time, resulting not only in speed but in economy.

Each section of track laid represents a new problem, and from time to time the writer has had to design special machinery in order to do the work quickly and economically. On Granville Street, the principal business street, though the paving was in good condition, about ½ mile of track was in such bad repair that it had to be renewed. The Board of Public Works allowed the company to abandon traffic on this street on condition that it would be restored in 22 days. As this period included 10 days for the last concrete laid to set, it meant that all work, except paving, had to be completed in 12 days. In order to be sure of carrying out the work on time, a progress chart, Fig. 20, was prepared, and was followed so closely that the work was finished 1 hour ahead of time.

The street was paved with wood blocks laid on a 6-in. concrete base, and each rail of the double track was carried on a concrete girder, 24 in. wide and 15 in. deep. The reconstruction of the track called for the removal and replacing of the following quantities: 5 100 sq. yd. of wood block paving; 10 200 ft. of rail; 1 800 cu. yd., or 5 100 sq. yd., of concrete; 600 cu. yd. of earth; the laying of 2 400 cu. yd. of concrete; ½ mile of double track; 850 sq. yd. of concrete flangeway blocks; and 4 250 sq. yd. of wood block paving. The quantity of concrete to be removed was so great, and the time allowed so short, that some mechanical means for breaking it up had to be devised, as the schedule called for the removal of about 32 sq. yd. per hour. The dipper arm was taken off the electrically-driven shovel owned by the company, and pile-driver leads, 20 ft. long, were hung from the end of the boom, being held in place by two braces running back to the main frame of the shovel. A 2 600-lb. hammer was specially cast, having a wedge-shaped cutting edge, about 8 in. deep, running parallel to the track. The cutting edge was placed in this way in order that it would not injure the remaining concrete in the street. This scheme worked excellently, and no trouble whatever was found in breaking up the concrete and adhering to the schedule. The concrete breaker, which would swing through 360°, was placed on one track, and on the other track was placed the derrick car, having a 60-ft. boom which would also

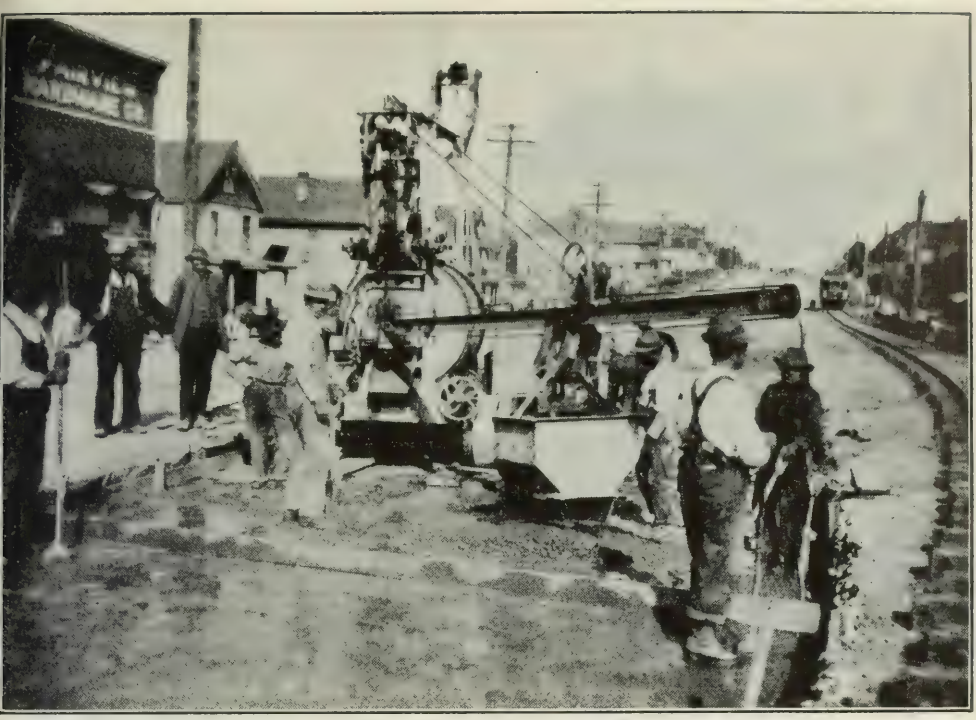


FIG. 16.—LAYING CONCRETE SLAB, BROADWAY, VANCOUVER, B. C.

FIG. 17.—LAYING AND SURFACING PERMANENT TRACKS, BROADWAY, VANCOUVER, B. C.

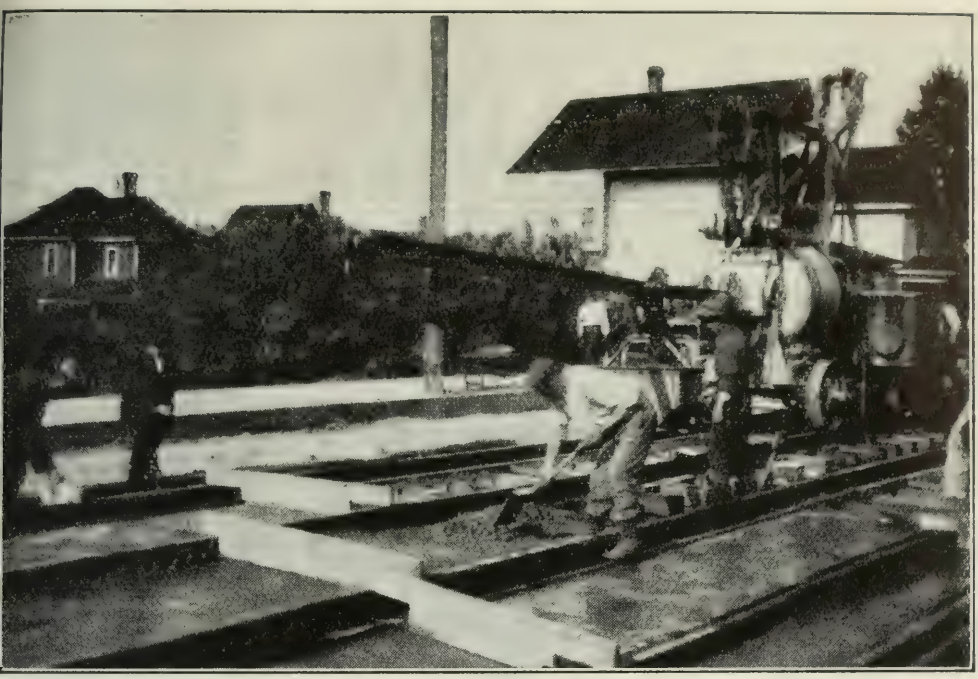
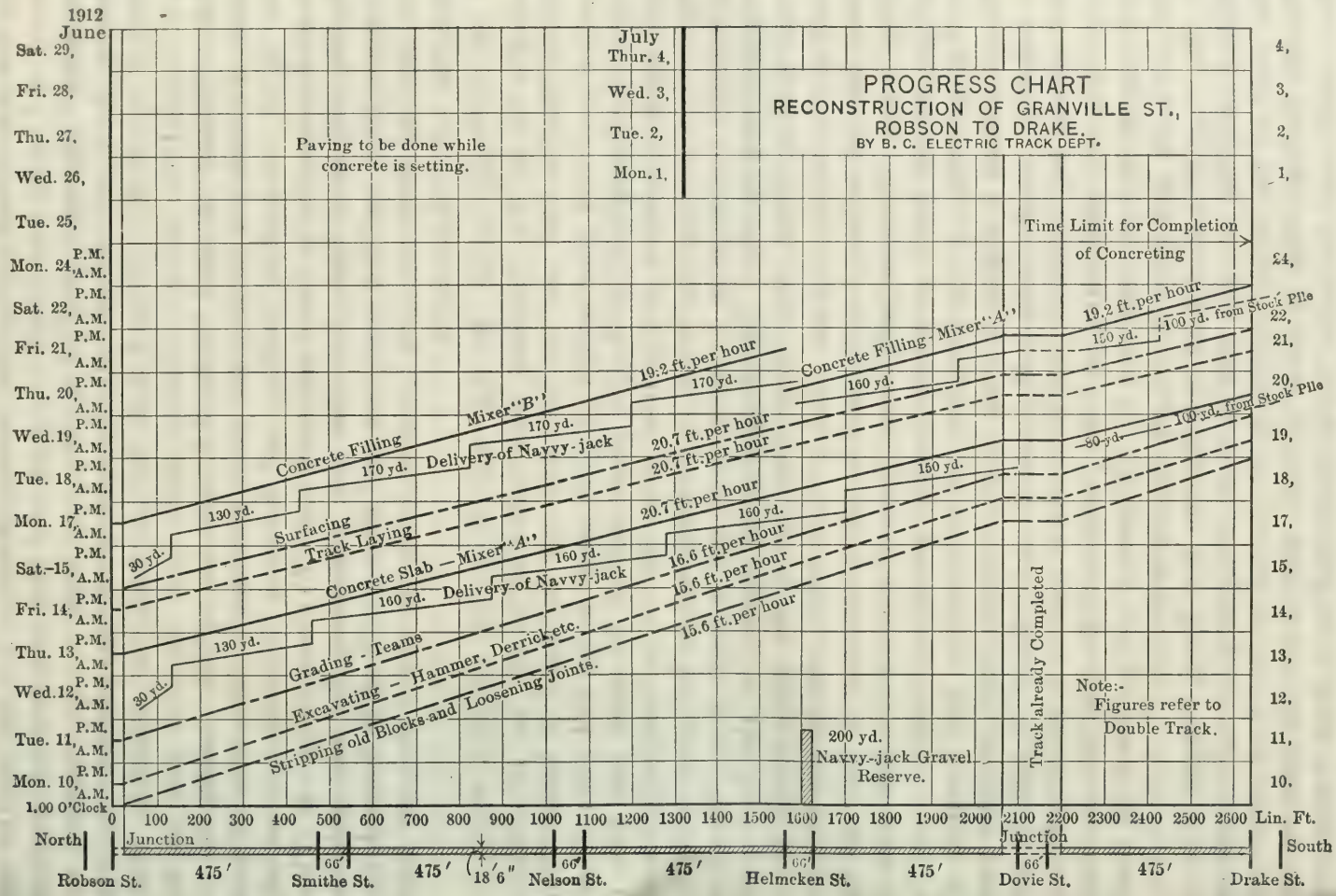


FIG. 18.—BRINGING CONCRETE TO UNDERSIDE OF PAVEMENT, BROADWAY, VANCOUVER, B. C. CONCRETE INSIDE THE RAIL LEFT LOW FOR 6-IN. GRANITE FLANGEWAY BLOCKS.

FIG. 19.—CONCRETE BREAKER, GRANVILLE ST., VANCOUVER, B. C.

Mr. Vorce.


PROGRESS CHART
RECONSTRUCTION OF GRANVILLE ST., ROBSON TO DRAKE.
BY B. C. ELECTRIC TRACK DEPT.
Paving to be done while concrete is setting.
1912
June
July
Sat. 29,
Fri. 28,
Thu. 27,
Wed. 26,
Tue. 25,
Mon. 24,
Sat. 22,
Fri. 21,
Thu. 20,
Wed. 19,
Tue. 18,
Mon. 17,
Sat. 15,
Fri. 14,
Thu. 13,
Wed. 12,
Tue. 11,
Mon. 10,
Thur. 4,
Wed. 3,
Tue. 2,
Mon. 1,
P.M.
A.M.
1.00 O'Clock
Time Limit for Completion of Concreting
Concrete Filling Mixer "B"
Concrete Filling Mixer "A"
Delivery of Navvy-jack
Surfacing
Track Laying
Concrete Slab Mixer "A"
Grading Teams
Excavating Hammer, Derrick etc.
Stripping old Blocks and Loosening Joints.
19.2 ft. per hour
20.7 ft. per hour
16.6 ft. per hour
15.6 ft. per hour
170 yd.
160 yd.
150 yd.
130 yd.
30 yd.
80 yd.
100 yd. from Stock Pile
200 yd. Navvy-jack Gravel Reserve.
Track already Completed
Note:- Figures refer to Double Track.
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500 2600 Lin. Ft.
North
Junction
South
Robson St.
Smithe St.
Nelson St.
Helmcken St.
Dovie St.
Drake St.
475'
66'
18'6"

Mr. Vorce. swing through 360°, and could move itself along the track. The larger pieces of concrete were picked up by the derrick and loaded on flat cars placed behind the concrete breaker, and the smaller material and earth were loaded in skips and then into wagons standing alongside the excavation (Fig. 19). As soon as the concrete was broken for a rail length, the concrete mixer and derrick car were backed up on the old track and the operation was repeated. Work was carried on in two 10-hour shifts, and the cost was as follows: Taking up wood blocks, 8.9 cents per sq. yd.; tearing up old track, 11 cents per ft. of single track; breaking up concrete, 40 cents per sq. yd.; excavation, loading, and dumping, 82½ cents per cu. yd.; removing material by teams and work train, 65 cents per cu. yd. These figures may seem high, but when one remembers the rate paid for common labor, and that teams are paid $8 per shift of 10 hours during the day, and $9.50 per shift for night work, the writer does not think that the cost was excessive.

Mr Polk. A. C. Polk, Assoc. M. Am. Soc. C. E. (by letter).—The point brought up by Mr. Tratman, with reference to the use of ordinary bricks tilted so that one end fits under the rail head, is noted. This method gave very poor results on some track which had already been placed in Springfield; in fact, a great deal of the work was done in this manner previous to the time when the writer commenced work there. It was found that teams and loaded wagons had great difficulty in turning out from in front of cars, and a great many wheels of vehicles were broken. The flangeway, as formed in the Springfield work, will, no doubt, wear down somewhat on the edges of the bricks next to the rail, but on one piece of work which was in use 1½ years there was no serious trouble, and no undue amount of wearing had taken place. In practically every type of construction the wear is greatest at this point, no matter how the paving is placed.

The crowning of the pavement, as criticised by Mr. Tratman, although making a slightly uneven contour in the street, does not seem to cause any trouble with traffic; wagons and vehicles are able to turn out without any difficulty, and the uneven contour is scarcely noticeable. The city authorities and the public both thought this construction a great improvement over the old type of placing the brick under the rail head, and were very much pleased with it.

The type of construction described by Mr. Howe, namely, 6 in. of ballast under the ties and then concrete on top as a paving foundation, was used by the writer in Mobile in placing a very busy cross-over. Reports indicate that it stood up very well under the constant and heavy traffic, and the writer has no doubt that, as a cheaper method of construction, this type answers very well, but he would not consider it as of a permanent nature.

Mr. Polk.

The decision to put in the type used in the Springfield work was arrived at from previous experience with cheaper types, where it was shown that in the long run they were far more costly than a permanent and rigid construction.

One thing to be noted in connection with the Springfield work was the adoption of a standard section of rail of the Am. Soc. C. E. pattern. This, of course, made the cost less than it would have been with a girder rail.

Mr. Howe's method, as used in Oakland, seems to the writer to be an excellent compromise, where it is not possible to put in a more expensive type.

It is noted that Mr. Mitchell has had very poor results with the concrete beam construction. The writer, however, has had quite the contrary experience where the concrete beam has been placed and has been reinforced with twisted steel rods, as in the type described.

The rail ends were not ground on the Springfield work. Although, no doubt, this is a most excellent additional precaution, there was no visible sign of any movement at the joints, after the first work at Springfield had been down 12 or 14 months. As a matter of fact, it was very difficult, indeed, to discover the joints at all.

All the bolts, as mentioned by Mr. Mitchell, were driven to an extremely tight fit. There was no trouble from cupping on any work of this class, some of it having been in service more than two years, not only at Springfield, but at other places. A great deal of the strength and rigidity of the Springfield joints was due to the type of plate used.

It is noted that Mr. Vorce also has had poor results with the beam type of construction, but he does not state whether he used a reinforced beam. The writer believes that this reinforcing has a greal deal to do with the character of the beam construction, and makes it a decidedly different proposition from a beam which has no reinforcement. There is no doubt that poor foundations would affect the failure of the beam, but reinforcement, properly placed and in sufficient quantity, would hold where a straight concrete beam of larger section would fail.

The writer does not agree entirely with Mr. Vorce in his statement that a hard, rigid, inelastic track is undesirable. As a matter of fact, as soon as a track foundation is put down, in a street paving in which there is any movement at all, the surrounding pavement commences to work, and in a very short time will be broken up and destroyed. The writer is inclined to believe that, in building in an expensive and costly pavement where there is considerable traffic, and where rebuilding is almost prohibitive, it is far cheaper in the long run to put down a solid, heavy roadbed.

The wooden ties would probably be satisfactory, but the writer is of the opinion that, if they are to be used in a permanent structure, it

Mr. Polk. would be desirable to place tie-plates on them, which would, no doubt, add to the life and reinforcement of the track.

The writer has mentioned the criticism as to the grinding of the rail ends, and although it may be that the matter is of more vital importance than he considers it, his experience on two previous construction jobs, of the character illustrated in the paper, did not show that planing off the rail was nearly as essential as the proper placing of the joints and of the concrete around the ties, with a big, heavy tie under the joint, also the additional reinforcement at this point.

Mr. Vorce's description of the work in Vancouver is certainly most interesting, and should be of considerable value to any one interested in this particular type of work.

The writer is very much pleased to have heard from the various members who have discussed his paper, and to have had other views than his own expressed in this matter.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1253

THE SANITATION OF CONSTRUCTION CAMPS.*

By Harold Farnsworth Gray, Jun. Am. Soc. C. E.

With Discussion by Messrs. R. C. Hardman, Spencer Miller, Frank E. Winsor, W. E. Britton, F. W. Austin, Harry G. Payrow, and Harold Farnsworth Gray.

A problem which often confronts the engineer or contractor engaged in work which requires the establishment of a camp is the keeping of labor steadily on the job in such physical and mental condition that each man puts forth his best efforts. This is especially true when work is prosecuted in new and unpopulated country, where life at its best must be rough and at times hard. The engineer in charge may have to carry on the work with a labor force depleted by sickness. As a result, many may leave permanently, and many of those who remain may be in such poor health that their physical efficiency is greatly impaired. A shifting or half-sick labor force cannot be efficient.

The writer has seen camps where a considerable proportion of the men were laid off because of intestinal disorders, due largely to poor sanitation, and where the efficiency of the force was reduced perhaps one-half. He has been in camps in malarial regions where the reduction in efficiency of the force was known to be at least one-quarter, and at times was probably greater. Such conditions represent a serious loss to the contractor, a loss which can be almost entirely eliminated by the observance of a few simple rules based on the present knowledge of disease prevention. In the fol-

* Presented at the meeting of December 18th, 1912.

lowing the writer proposes to set forth briefly some of the principles of sanitation which are applicable to camp conditions, in the hope that he may convince the Engineering Profession that good camp sanitation is an economy, and not an expense.

Camp Location.—The matters of site and soil conditions are not of prime importance. Even under the poorest natural conditions, good health may be maintained in the camp if the proper sanitary measures are taken, but the better the natural conditions, the fewer and less costly will be the sanitary measures required.

Preference should be given to an even surface with a slight natural slope. An even surface promotes convenience and accessibility, and a slight slope gives a quick run-off for possible rain water. In cool climates a sunny exposure should be selected, and, in hot climates, the shady side of a valley, if possible. Low places, swamps, and wet areas should be avoided, as these may be breeding places of malaria-bearing mosquitoes. A light, porous soil is an advantage, not that disease lurks in the emanations from damp ground, but because of the added comfort of dry quarters.

Housing.—The form of housing will depend largely on the duration of operations and on transportation facilities. Whether tents, portable houses, or the more permanent wooden bunk-houses are used, ample provision for the free entrance of air and sunlight should be made. If wooden bunk-houses are used, the window space should be ample, and the windows swung inward on hinges at the side of the frames. In warm climates, in summer, and especially in fly and mosquito regions, the doors and windows should be screened, as described later.

One or more able-bodied men, as required, should be assigned to janitor duty. Each day all quarters should be opened as widely as possible for several hours during the morning for ventilation, and thoroughly cleaned by the janitor. Dirty conditions and violations of the camp rules should be reported by him, and the offender warned, disciplined, or discharged, according to the magnitude and frequency of the offense. With proper cleanliness, disinfectants are not required, except in unusual circumstances.

In cold climates, and during the winter, stoves (of a type which carries the products of combustion out of the room) should be provided, and fuel supplied to the men.

Mess-House.—The mess-house and kitchen should be especially ell ventilated, but all doors, windows, and other openings should be creened with a copper or bronze wire of No. 18 mesh screen. Doors hould be provided with springs to keep them closed. In order to revent the entrance of insects around the edges of screens, the rames should close flush against wood strips nailed to the casing.

All reasonable means for obtaining cleanliness in the preparation nd handling of food should be provided, and the kitchen and mess-ouse should be thoroughly cleaned after each meal. It is desirable hat the various sources of food supply be known, where possible, nd only such foods accepted as are wholesome and are produced nd marketed under reasonably clean conditions. This is no doubt ifficult under average conditions, but is worth considerable effort. s far as possible, the diet should be suitable to the season. No person fficted with a communicable disease should be allowed to work in ny capacity in the kitchen or mess-house.

Garbage.—Metal receptacles, provided with tight metal covers, hould be supplied and used for the kitchen wastes, both solid (gar-age) and liquid (slops). In camps which are to be maintained or a period of two weeks or longer, all garbage and slops should be estroyed by incineration, as any other form of disposal will probably ermit fly-breeding. In no case should either garbage or slops be emptied n the ground, even at a considerable distance from the camp. Fig. 1 hows a simple and inexpensive garbage and slops incinerator, de-gned by Maj. Paul F. Straub, G. S., U. S. Army, and is given ere by his permission. This incinerator was used successfully at he maneuver camp at San Antonio, Tex., in 1911. With proper ttention, it will destroy 100 gal. of slops and 23 cu. ft. of garbage n 12 hours, with a fuel consumption of $\frac{1}{16}$ cord of wood. With the rdinary attention which would be given in a construction camp, acking in rigid military discipline, the fuel consumption would be reater, but $\frac{1}{8}$ cord per day should be ample for the destruction of ll garbage and slops created in the preparation of food for 100 men.

The liquid slops are evaporated by being poured slowly along he walls of the incinerator, and the garbage is placed on top of the uel. If rock for the construction of the incinerator is not avail-ble, a trench of the same form and capacity may be dug in the ground,

and operated in the same manner. Great care should be taken to see that the garbage is thoroughly consumed; otherwise, fly-breeding may ensue if the incinerator is not used constantly.

Latrines.—Indiscriminate defecation and urination in the vicinity of the camp should be prohibited, and a sufficient number of latrines constructed at convenient places. A two-hole seat with a urinal

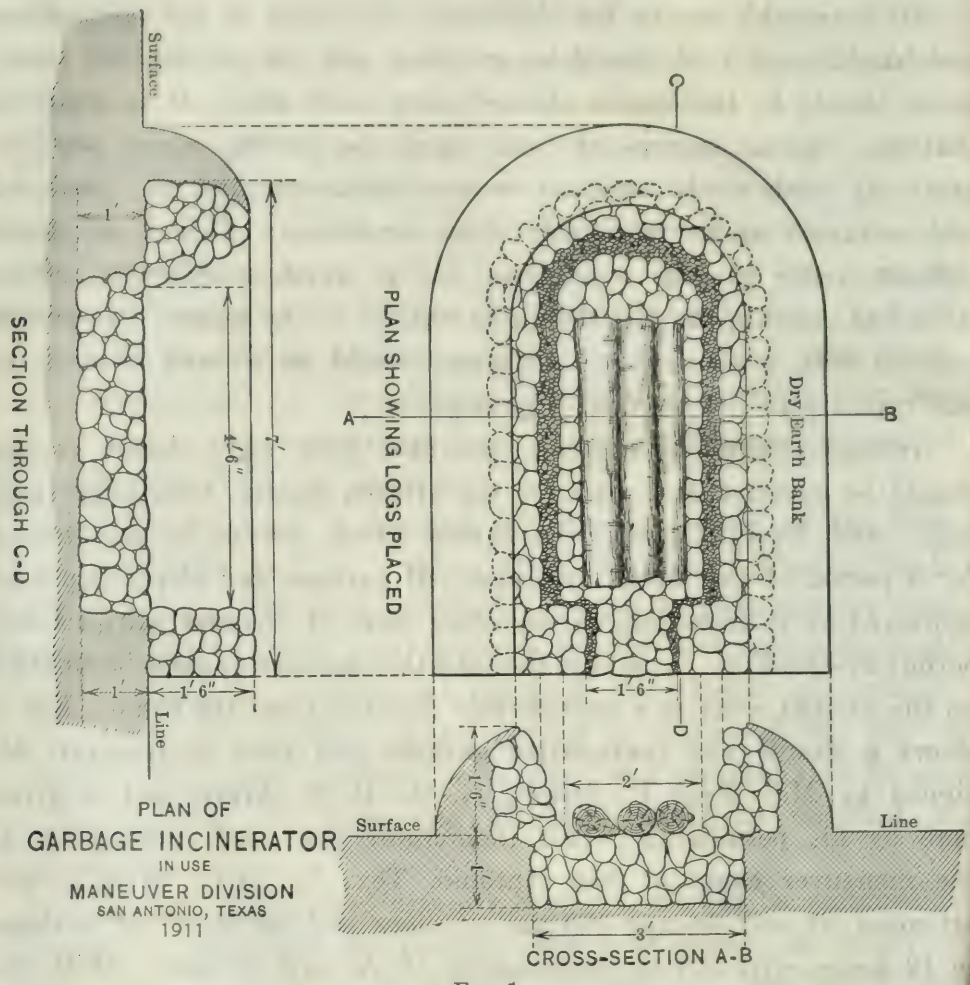
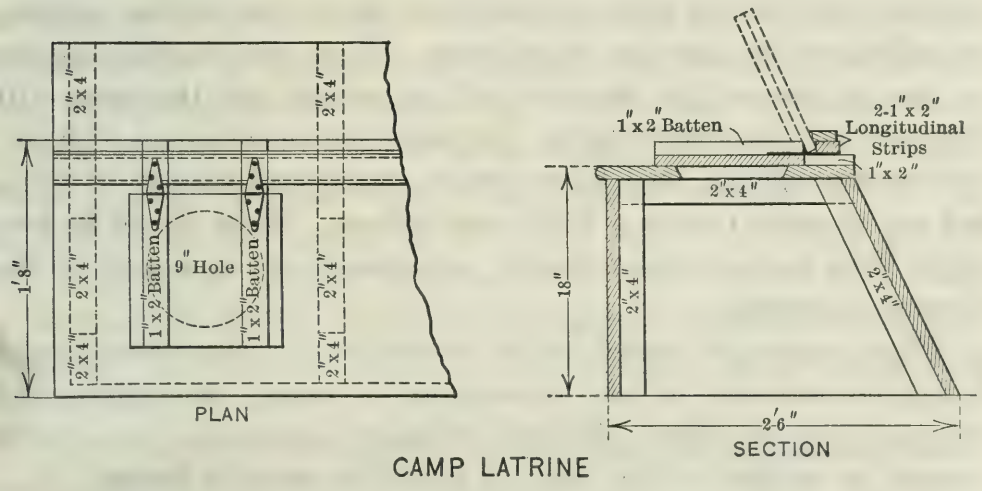


FIG. 1.

trough is a good standard for the average camp, and the latrines should be placed so that the maximum distance from any house or tent to the nearest latrine does not exceed 100 ft. With easy accessibility of latrines, promiscuous defecation is more easily discouraged.

The latrines consist of three parts: the pit, the box, and the shelter. The pit should be 6 ft. or more in depth, and in section slightly smaller than the box. The box is shown by Fig. 2 (also by permission of Maj. Straub). It should be made of tongued-and-

grooved boards, as ordinary lumber tends to warp and shink, so that it is difficult to keep the box fly-tight, reliance being placed on the tightness of the box to prevent the access of flies to the contents of the latrine. The covers of the holes should drop back automatically into place over the holes when the seat is not occupied. In summer no roof is provided for the shelter which, for privacy, is built around the latrine, as the direct action of sunlight and air is desirable as a disinfectant and deodorant. During the winter a roof may be provided, and during rains, at other seasons of the year, a tarpaulin should be used.



CAMP LATRINE

FIG. 2.

A metal urinal trough may be attached to the side of the latrine shelter, discharging into the pit through the box *via* a trap. The pit should be disinfected daily by being burned out with oil or oil-soaked straw. The urinal trough should be flushed with water and limed, and the interior of the box and the pit should be limed after being burned, to deodorize the latrine. Cleanliness in the use of latrines must be strongly insisted on, especially if the laborers are of foreign nationality.

Stables.—The stable, especially the stalls, should be cleaned out thoroughly each day, and the manure removed to a point at least ½ mile from the camp and burned with oil. In a hot, dry climate, it will suffice to remove the manure to this distance and spread it out thinly on the ground; either method will prevent fly-breeding. Under average conditions, horse manures are the favorite breeding places of flies. From estimates based on actual counts of represen-

tative samples, it has been shown that a manure pile weighing 1 000 lb. may contain upward of 450 000 fly maggots at the end of 4 days' exposure. Therefore, unusual care should be taken of manures, and the stables kept clean. There is no substitute for cleanliness; treating the stalls and manure with oils, chemicals, and poisons is in general more expensive and less effective than cleanliness and the proper disposal of manure.

Water Supply.—An endeavor should be made to obtain a pure water supply. Where the water is known to be polluted, it can be sterilized by calcium hypochlorite (chloride of lime or bleaching powder), used at the rate of from 10 to 40 lb. per million gallons, depending on the intensity of pollution. When used in this amount no taste or odor of the chemical will be noticed, and the water will be potable and practically sterile. For more permanent camps with polluted water supplies, sand filters can be constructed cheaply of wood, and can be made to give a fairly good effluent. Wells should be protected from surface contamination, and should not be placed in the vicinity of latrines.

Where camps are located on the water-shed of a city water supply, it will be necessary to take precautions to prevent the pollution of the supply by the camp. All such necessary precautions are usually covered by the laws of the State in which the camp is located.

Baths and Laundry.—In many respects personal cleanliness is one of the most important factors in health preservation, for the reason that disease is largely transmitted by contact, from person to person. It is advisable, therefore, that facilities for keeping personally clean be provided, and the men encouraged to make use of them. Shower baths can be easily and cheaply constructed. A simple and effective shower can be made from a 2-ft. length of 1-in. pipe, capped; along the lower side of the pipe are bored three parallel rows of $\frac{1}{16}$-in. holes set $\frac{3}{8}$ in. apart. The pipe projects horizontally from a tee in the vertical supply pipe, at about 8 ft. above the floor. The latter should be of wood with open joints. The bath should be provided with a shelter, and be without a roof in summer.

Laundry equipment and supplies, such as tubs, washboards, and soap, should be furnished gratis.

Rubbish.—Unless cared for, considerable rubbish tends to accumulate in any camp. It should be collected daily by the janitor, removed

to a specified place, and burned. Everything which tends toward neatness and cleanliness should be encouraged.

Flies.—The principal breeding places of flies, namely, manure piles, garbage heaps, and latrine pits, have been mentioned. They may also breed in any collection of decomposing organic matter, no matter how small. The fundamental theorem of fly control is cleanliness combined with proper destruction of waste products. Where destruction is not feasible, the rendering of such wastes unsuitable to fly-breeding, or inaccessible to flies, may be substituted.

Under midsummer conditions, flies will develop from egg to adult in from 10 to 14 days. The adult female lays from 75 to 125 eggs on the surface of the breeding material, and there may be several such layings during the average life of the insect. The eggs hatch in from 12 to 24 hours; the larvæ or maggots feed from 4 to 7 days, crawl away to the lower part of the breeding material, or into the adjacent earth, and pupate; in the pupal stage they remain quiet 4 days or more, and emerge full-grown flies. Fig. 3, given by courtesy of William B. Herms, Assistant Professor of Applied Parasitology, University of California, shows the four stages in the development of the common house-fly.

The rapidity of development is dependent on temperature, being accelerated by warmth and retarded by cold. Owing to the fairly constant warmth of decaying organic matter (for example, manure), the larval period is very nearly constant. The pupal period, being spent away from the decaying material, shows the greatest variation, and, in winter, may be lengthened to several months.

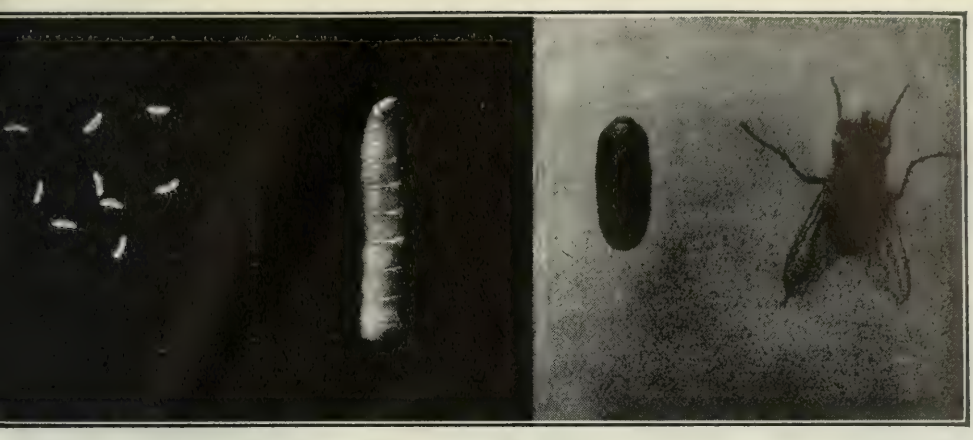
Owing to the fact that the fly maggot is very resistant to poisons and insecticides, it will be found cheaper and more effective, as previously stated, to destroy or remove the breeding material, than attempt to treat accumulations with poisons.

It has been proven conclusively that under conditions such as are usual in camps, flies may be gross transmitters of such diseases as typhoid, dysentery, and various intestinal disorders. During the Spanish-American War, when more soldiers were killed by disease than by bullets, large numbers of flies were often observed on the food in the mess-tents, the bodies and feet of many of them being whitened by the lime in the latrines which they had recently visited.

It may be stated almost as an axiom that the sanitary condition of a camp varies inversely as the numbers of flies present.

Malaria and Mosquitoes.—It is now conclusively proven that mosquitoes of a certain type are the sole transmitters of malaria. This disease is most prevalent during the warm months of the year, when the largest amount of construction work is carried on. In malarial regions the lost efficiency of the labor force from this disease is often 25% and more. Even where malaria-bearing mosquitoes are not present, and those of the common type are numerous, it is advisable to take measures for their destruction, as a night disturbed by these insects does not put a man in the best condition for a hard day's work. As previously mentioned, the doors and windows of wooden houses should be screened and, in mosquito districts, if tents are used, bed-nets supported by frames should be supplied.

The two types of mosquitoes, the *Anophelines,* or malaria transmitters, and the *Culicines,* or common mosquitoes, are shown in various stages of their life history on Fig. 5. The differences in the two types are easily recognized in three of the four stages. The *Culicines* lay their eggs on the water surface in boat-shaped masses of from 250 to 750 eggs; the *Anophelines* lay their eggs singly or in irregular clumps. In the larval or wriggler stage, passed in water, the *Culicines* project a posterior breathing tube to the surface, the remainder of the body hanging head downward at an angle with the surface; the *Anophelines* breathe while lying parallel to, and with every segment touching, the surface. The differences in the pupal or tumbler stage are not sufficiently well marked to be easily distinguished by the unaided eye. In the adult stage, two methods may be used to distinguish the types: The resting attitudes are different; an *Anopheline* resting on a plane surface holds the posterior part of the body away from the surface, the line of the body thus making a distinct angle of from 25 to 55° with the surface, whereas the *Culicine* adult rests with the line of the body parallel to the surface, or in a humpbacked position. The best method of distinction in the adult stage is based on the mouth parts, which can be seen with the unaided eye, but can be observed better with a low-power magnifying lens. In the *Anopheline* females, the palpi (the two appendages immediately on each side of the central proboscis or beak) are approxi-



(*a*) (*b*) (*c*) (*d*)

FIG. 3.—DEVELOPMENT OF THE COMMON HOUSE FLY.
(*a*) EGGS. (*b*) LARVA OR MAGGOT. (*c*) PUPA. (*d*) ADULT FLY.

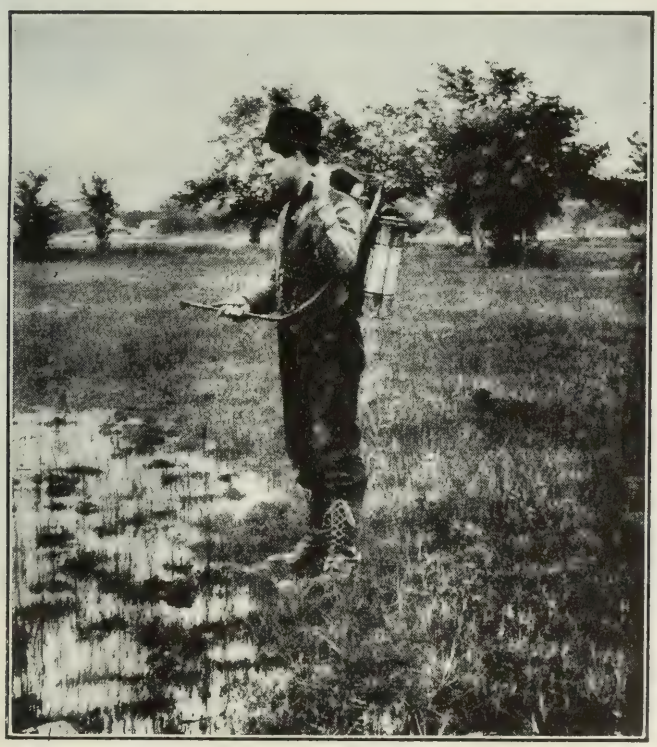


FIG. 4.—KNAPSACK SPRAY-PUMP IN USE, SPRAYING OIL ON A TYPICAL ANOPHELINE BREEDING PLACE.

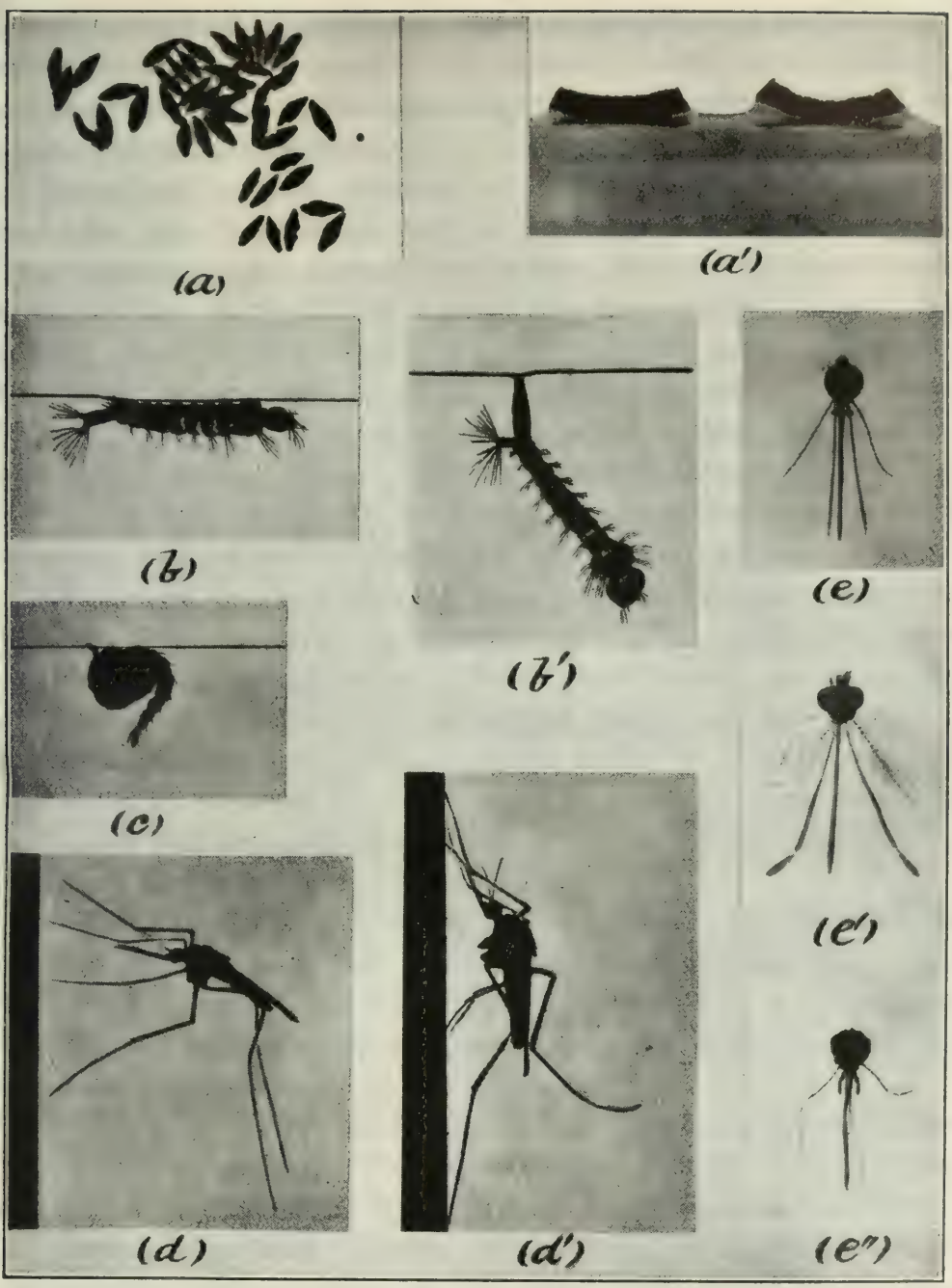


FIG. 5.—DEVELOPMENT OF THE MOSQUITO.

(*a*) ANOPHELINE EGGS (AFTER HOWARD). (*a′*) CULICINE EGG-BOATS.
(*b*) ANOPHELINE LARVA. (*b′*) CULICINE LARVA OR WRIGGLER.
(*c*) ANOPHELINE PUPA OR TUMBLER.
(*d*) RESTING ATTITUDE OF ANOPHELINE ADULT.
(*d′*) RESTING ATTITUDE OF CULICINE ADULT.
(*e*) HEAD AND APPENDAGES OF FEMALE ANOPHELINE.
(*e′*) HEAD OF MALE MOSQUITO, EITHER TYPE.
(*e″*) HEAD AND APPENDAGES OF CULICINE FEMALE.

mately as long as the proboscis, while in the *Culicines* the palpi are shorter than half the length of the proboscis. The males, which do not suck blood, can be distinguished, if found, by the feathered or plumose antennæ or feelers.

Mosquitoes breed in stagnant or quiet water, the *Anophelines* preferring clear, fresh water standing but a few inches on grassy land. The *Culicines* are not as fastidious in their choice of breeding places, often thriving in the foulest water. The first principle of mosquito control, therefore, is the removal of all stagnant water. This may be effected by drainage, by filling in small low spots which cannot be readily drained, by emptying vessels which contain useless water, and by screening all others. Where drainage or filling in is not practicable, as will usually be the case in construction camps, oil or chemicals may be used to destroy the mosquito larvæ.

Oil sprayed on the surface of water which is breeding mosquitoes will kill them effectively by shutting off the air supply, the larvæ and pupæ dying by suffocation. The oil should have a specific gravity of about 30° Baumé, and may be applied conveniently and economically by a knapsack spray-pump, as shown in Fig. 4. Small ponds may be treated by throwing in several handfuls of cotton waste soaked in oil, which will gradually feed a film of oil on the surface for some time, and may be renewed at intervals.

Under midsummer conditions, the development of the mosquito requires from 10 to 14 days; in the spring and autumn, this period is somewhat longer. To prevent breeding, therefore, it is necessary to apply the oil by spraying once every 2 weeks in summer, and about once every 3 weeks in the spring and fall.

Where the water is full of vegetation, it is advisable to treat it first with copper sulphate (1 to 50 000) and then with oil. A preliminary copper sulphate treatment will be found an advantage where poisons or larvacides are used. Several poisons, such as nicotine sulphate, phinotas oil, and emulsions of a modified crude carbolic acid, have been used for mosquito destruction with varying success, but on the scale of operations which would be usually attempted for the protection of a construction camp, oil will be found to be cheaper and more effective.

It will seldom be necessary to carry on anti-mosquito operations beyond a distance of 400 yd. from the camp, and not all the water

in this territory will be found to be breeding mosquitoes. Only water which is actually breeding them need be treated.

Medical Treatment.—While the subject of medical treatment of the sick does not properly come under the provisions of camp sanitation, it is advisable to consider it briefly here, on account of its relation to the control of disease. The character of such treatment will depend on the size of the camp and the country in which the work is carried on. For small camps, in sparsely inhabited regions, a medicine chest of standard remedies, with bandages, liniment, and first aid to the injured materials, should be provided, to be administered gratis as needed. In malarial regions, in addition to screening and mosquito destruction, a daily prophylactic dose of quinine should be urged on the employees. Where small camps are within easy reach of a town, arrangements should be made with a physician to call at regular times, and as needed in emergencies, and, in addition, the medical and first aid equipment should be provided for minor complaints.

Where several camps are working a large number of men, directed from a central field office, it is advisable, and usually practicable, to install a field hospital with a resident physician, who, in addition to his hospital duties, makes periodic inspections of the camps. The expense of the hospital is usually provided for by nominal deductions from the pay of each employee, which entitles him to free medical attendance if sick or injured. This is often done on works of magnitude, an example with which the writer is familiar being the Los Angeles Aqueduct.

In any case, an employee suffering from any form of communicable disease should at once be laid off and isolated from the others, to prevent as far as possible the further spread of the disease.

The Value of Good Sanitation.—In its broadest sense, sanitation includes all methods and procedures necessary for the prevention of the spread of diseases. In its practical workings, it is limited to certain fields, which are not very well defined, owing to the interrelations of various factors in disease prevention. The procedure in disease prevention may be arranged roughly in three main divisions: First, the control of the source of infection, practically always a person sick with the disease; second, the control of the carrier or

means of distribution of the disease from one person to another; and third, the protection of the well persons individually.

The first division falls largely in the domain of the physician, though the engineer, by installing works for the prompt removal and purification of the infected discharges of the patient, plays his part here. The control of the carrier of infection, such as food, water, sewage, insects, etc., comprises the field of sanitation as usually defined, and in this field the engineer, chemist, bacteriologist, entomologist, and others, play the chief part. Personal hygiene largely covers the third division. All three divisions are links in a chain which is no stronger than its weakest link. Under camp conditions, however, it is not practicable to take the measures which are essential in an urban community, and it is not possible to pay much attention to personal hygiene. The bulk of disease prevention, therefore, must fall in the second division.

There are few engineers who are not familiar with the usual conditions in camps. Foul-smelling, open latrines swarming with flies, dirty and poorly ventilated quarters for the men, vile food, filthy mess-tent, and swarms of flies on and in everything, are the average conditions. Such being the case, it is well-nigh impossible for any man to remain in good physical condition, and it is the laborer's physical condition which largely determines the amount and character of his output. If a part of the labor force is weakened by ill-health, just that much less than the maximum possible work can be obtained. The contractor expects the maximum for his greatest profits; if he gets less than the maximum, he sustains a loss. A reduction of 25% in the efficiency of the labor force is not unusual. If we consider a camp of 100 men at an average daily wage of $2, this daily loss amounts to $50, and in a 30-day month would reach a total of $1 500. The additional cost of good sanitation above the amount spent for the poor sanitation which is at the bottom of this loss, would probably not exceed $250 per month, leaving a net profit, chargeable to good sanitation, of $1 250. The average saving would undoubtedly be less than this amount, but the writer does not believe that he has based this estimate on an extreme case. However, be the saving much or little, good sanitation in a camp is emphatically an economy, not an expense. Aside from ignorance, perhaps the chief

reason that camp conditions are usually so bad is that the contractor can actually see in dollars and cents the money expended for good sanitation, while the saving cannot be expressed so accurately or in such tangible form. Nevertheless, the economy can be shown at least approximately in every case, and in extreme cases may represent the difference between a profit and a loss on the contract.

DISCUSSION

Mr. Hardman.

R. C. Hardman, Assoc. M. Am. Soc. C. E. (by letter).—This paper is one of very timely interest, its only fault being its brevity. The writer has had occasion to witness the good effects of camp sanitation, as well as to install certain sanitary appliances in a camp of 4 100 men, under unfavorable climatic and drainage conditions. The subject is one which deserves to be expanded, and should receive the careful attention of the Profession.

In some camps which the writer has seen, the conditions approach those which must have existed in the Middle Ages, and now exist in the less enlightened portions of the world. In the provincial districts of the Philippines, for instance, Americans are horrified to see practically all human waste consumed by hogs kept in the back yards of nearly all the native shacks; yet they view with apparent equanimity the promiscuous deposition of fecal matter around a camp wherein the services of our porcine friends are not available.

The writer once met an American in Hong Kong who solemnly swore total abstinence from fish because he had seen in Canton, China, a fish pond wherein the fish were fed, in part, from a privy built over the edge of the pond. Yet we, at home, empty our raw, untreated sewage into our rivers from which fish are taken for the table, and in thousands of instances privies are built over streams and ponds, as in the case of the Buddhist monastery in Canton; and yet "the pot called the kettle black." The Cantonese did not use the pond for a water supply; we do use our rivers for that purpose.

To carry the comparison still farther: The writer has seen in Chinese cities huge tubs along the streets in use as "public comfort stations." In a great many cases, these "stations" overflow into the streets; yet this is not much worse than conditions in some construction camps where latrine trenches are dug, left open to the light and air, and flies, and allowed to fill up without destructive or disinfective treatment of any kind.

We think of the Chinese race and nation as a very filthy one, but the filth is only a matter of comparison. The Chinese have yet to be educated up to a high sanitary standard, while we have to be educated to the pre-eminence of health over dollars.

The writer has brought this subject to the attention of the Profession in a short article* containing rules for the use of the various appliances described and illustrated. They differ slightly in form from Mr. Gray's, but cover the same ground. The method, however, is of minor import.

Perhaps the greatest arguments for camp sanitation are found in the records made in the Canal Zone, in Cuba after the Spanish-

* *Engineering News*, October 17th, 1912.

Mr. Hardman. American War, in the various maneuver camps of the United States army in recent years, and, notably, in the work of the Japanese Medical Corps in the war with Russia. One has but to compare these with the history of the French Company at Panama and the disgraceful record of the U. S. Medical Corps during the Spanish-American War, at Chickamauga, Jacksonville, and elsewhere.

Mr. Gray dwells only on the baser financial side of "the value of sanitation." Of course, this is the only thing which will appeal to the average camp executive, but should we not, as engineers, also be moved by the more altruistic motive of helping the ignorant foreigner, who makes up the bulk of our camp population, to attain, by example and precept, the standard of living of the Americans with whom he is to become assimilated? Do we not owe it as a debt to the human race?

Mr. Miller. SPENCER MILLER, M. AM. SOC. C. E.—Mr. Gray's paper is interesting and valuable.

A Government contractor, building a lock and dam on one of the Western rivers, recently informed the speaker that an outbreak of malaria in his camp had nearly cost him the profit on his contract. Malaria and malarial fever disorganized his camp so completely that he found it practically impossible to keep a full force of men at work. New men arriving to take the places of sick ones would shortly succumb to the disease; others fled from the camp. It was only after this had continued for a considerable time that the contractor sought relief from a sanitary expert. The outbreak of malaria was attributed directly to the presence of the malarial mosquito. The camps were thoroughly fumigated and screened. The sick were properly guarded against the mosquitoes. The immediate vicinity of the camp was cleared up. The rank growth was mown down. Ditches were dug and drainage operations were carried out thoroughly. Every receptacle for holding water was either emptied, destroyed, or oiled regularly. Shortly the trouble disappeared. The camp became a healthy and a profitable one. All of which illustrates that it pays to be sanitary.

Mr. Winsor. FRANK E. WINSOR, M. AM. SOC. C. E.—This paper is a timely contribution on an important subject relative to which very little has appeared heretofore in the publications of the Society.

The construction and sanitation of camps for the temporary housing of men have, generally speaking, received little scientific consideration among those responsible for their creation, and the author's suggestions are so materially in advance of common practice as to be considered by many, perhaps, revolutionary.

The proper sanitation of construction camps depends on a variety of considerations, and involves many diverse problems. The character of camp construction should depend on seasonal variations of climatic

conditions, the length of time which the camp will be occupied, the nationality and intelligence of the occupants, the character of the country in which the camp is located—particularly whether in a populated or non-populated district—and other less important considerations. Mr. Winsor.

The following discussion is mainly from the viewpoint of camps located in more or less populous districts, the occupancy of which extends over more than one working season. The methods of camp construction and sanitation described have been used successfully along large portions of the Catskill Aqueduct, a structure some 90 miles in length, extending from the Catskill Mountains to New York City, and now nearing completion. The work and methods outlined are both practical and reasonable, and are believed by the speaker to be adaptable, with suitable modifications in detail, to substantially all conditions. The subject may be conveniently treated under the following headings:

1.—Location of Camp.—Generally speaking, camps should be located on high, dry ground, preferably sandy; and, where located on a drainage area tributary to a domestic water supply, a site remote from watercourses should be selected.

2.—Water Supply.—A water supply of suitable quality and quantity for drinking, bathing, and washing must be provided, and, if the camp is to be used for any considerable period, the supply should be properly piped under pressure to the various buildings. A ground-water supply from deep wells, protected from surface pollution, is generally the most satisfactory, although under some conditions it may be allowable to use surface springs or surface streams, if their purity can be properly and continuously safeguarded. Dug wells, open at the top, are almost always polluted, and it is seldom safe to use water from streams and lakes, particularly in a territory at all populated, without proper purification. It is important, especially with sources other than ground-water, to make frequent analyses in order to secure a continuous suitable quality of the supply.

3.—Food Supply.—An intelligent selection and inspection of all food products, particularly milk and such articles as are used without cooking, is important.

4.—Waste Disposal.—The most efficient method of disposal of human excreta, including urine, is by incineration, and it has been found entirely practicable to collect and incinerate such wastes from camps of both large and small size. Various types of incinerators have been used, including several small, portable ones with seats and urinals attached; but, generally speaking, with camps of considerable population, with several years occupancy in prospect, it has been found more economical and practical to build special incinerating furnaces, properly housed, and to collect all the fecal matter in tight cans, which are placed under the seats in the closets and changed daily. Suitable closets for housing the cans, protected from flies by

Mr. Winsor.

thorough screening, should be provided throughout the camp and along the line of the work. The speaker is of the opinion that, under all conditions, incineration is much to be preferred to the latrines suggested by the author.

With the fecal matter disposed of, the problem of the other camp wastes is materially simplified. Garbage should be collected and burned daily. Stable manure should be disposed of daily, preferably by incineration, although it may be allowable, in agricultural districts, to permit its use on land, if it is hauled a sufficient distance from the camp to make sure that any flies which might breed in it cannot reach the camp area. Liquid wastes from the camp, other than those previously mentioned, may be taken care of in a variety of ways, depending on local conditions. If the camp is located in a drainage area tributary to a public water supply, particularly if the drainage from the camp enters such water supply near an intake, extreme precautions are necessary. In the case of camps located on the drainage area of Croton Lake (the main source of supply of that portion of New York City which includes the Boroughs of Manhattan and the Bronx), the following precautions were taken: The camp was surrounded by intercepting ditches, and the rainfall on the area inside these ditches was collected in a settling basin of sufficient capacity to hold the rainfall from substantially all storms. From this settling basin the water was run through sand filters. These sand filters also filtered the wash-water from the camp, this water being collected by a proper sewerage system. The effluent from these filters was afterward dosed with chlorinated lime, so that a substantially sterile effluent was finally obtained—generally much better than the natural flow of the stream into which it emptied. In case of camps located in polluted drainage areas not used for domestic water supply, the omission of filtration for surface run-off from the camp and the substitution of septic tanks or broad irrigation for wash-water, etc., may be permitted.

5.—Character of Construction of Buildings.—Ample provision for air space and ventilation should be made in the buildings, and a minimum allowance of 400 cu. ft. of air space per occupant is believed by the speaker to be none too much. Proper heating in the winter season and suitable lighting should be insisted on. The mess-house should be detached from the other buildings. Hospitals of proper size and suitable equipment should be provided, and a building which may be used as an isolation hospital should be in readiness in case of contagious diseases.

6.—Medication.—A physical and medical examination of applicants for employment is very desirable, and may avoid much subsequent trouble. The continuous attendance of a properly qualified physician, to take charge of the sanitation of the camp and the health of the

occupants, is very important, particularly in camps housing a considerable number of men. Mr. Winsor.

7.—Protection from Flies.—One of the most important problems in sanitation is to prevent, as far as possible, the breeding of flies, which are believed by the speaker to be one of the greatest pests in existence. The daily collection and disposal of all camp wastes, in most of which flies may breed, should be rigidly insisted on, and all quarters for the men, and, in fact, all buildings in the camp, should be properly screened.

8.—Fences and Lighting.—As extreme precautions, high fences of a man-proof type entirely surrounding the camp, with one or more openings which can, when desirable, be placed under proper guard, and illumination of the camp area at night, have been found very effective as aids in securing compliance with sanitary rules.

9.—Police.—An efficient police force, while not essential, is a valuable adjunct in connection with securing the best results.

There is no doubt in the speaker's mind as to the economy of proper camp sanitation. It has been demonstrated as both possible and practicable to eliminate substantially the scourge of typhoid fever and other contagious diseases, and to secure sanitary conditions in temporary camps in advance of those found in most established communities. There is, no doubt, a great increase in the individual efficiency of men employed under proper sanitary conditions, and a large gain also due to non-interruption of the work on account of sickness among the force. The restriction as to time is very important in nearly all construction work, and expenditures for adequate camp sanitation, which should never exceed a small percentage of the cost of the work, will almost invariably be returned several-fold in economy resulting from increased efficiency of the force, earlier completion of the work, and other indirect and less tangible benefits. The safeguarding of the health of the surrounding country is also a most important factor in work located in populous districts.

The efficient enforcement of sanitary rules is a very difficult matter with the class of labor ordinarily employed on construction work, and the immediate discharge of employees who are discovered violating these rules has been found to be the most effective means of securing their enforcement. A continuous campaign of education is also of great value as an aid to securing the best results.*

W. E. Britton, Esq.† (by letter).—The importance of camp sanitation has long been overlooked by both engineers and contractors, doubt- Mr. Britton.

* A very complete and able paper by Andrew J. Provost, Jr., Sanitary Expert, entitled "Sanitary Problems of the Board of Water Supply," outlining the methods and results of sanitation along the line of the Catskill Water-works, the new water supply for New York City, was presented before the Municipal Engineers of the City of New York, and is published in the *Proceedings* of that Society for 1911.

† State Entomologist, New Haven, Conn.

Mr. Britton. less for the reasons given in Mr. Gray's admirable paper. Recent discoveries, which show how many communicable diseases are transmitted by insects, render strict rules and regulations imperative, and sanitary precautions and appliances essential. As regards the laborers, ignorance is their chief excuse, but this is not so with the engineer, who should also be a sanitarian, for prevention is better than cure.

The construction of the Panama Canal was primarily a problem of sanitation, and could not have been solved from other standpoints. The average contractor is probably more easily moved by a desire to further the progress and profits of his work than by humanitarian and philanthropic motives. For this reason, an appeal for sanitation on the grounds of increased efficiency of his men and to prevent economic loss seems to be wise. A few damage suits for criminal negligence in extreme cases might have a salutary effect. Of course, there are many construction, lumber, and mining camps located far from other human habitations, and, in such cases, a neglect of sanitation affects only those connected with the camps. On the other hand, there are construction camps located on the outskirts of many cities and villages, and here a neglect of sanitary measures imperils the lives and health of thousands, though, in most cases, such camps would be within the jurisdiction of the health officer.

After the resident physician, the engineer is probably the most important officer concerned with the establishment of construction camps, and it is to him that we must look in the future for a large degree of improvement in camp sanitation. If he has a knowledge of the dangers and will assume the responsibility of avoiding and remedying them, we are making real progress. It is gratifying that papers of this nature are presented before the Society.

Mr. Austin. F. W. Austin, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Gray is to be congratulated for the manner in which he has presented this subject. The proper sanitation of camps should appeal to many construction engineers and contractors, and it is to be hoped that better sanitary conditions may be obtained. Any method or system, however, to appeal to the contractor, should consist of appliances which are easily and economically obtained. For example, Mr. Gray stipulates that copper or bronze screens should be used to protect the various openings, such as windows, doors, etc. The contractor is usually situated where copper or bronze cloth is not readily or economically obtained, and it is not quite clear to the writer why common iron screens will not answer the purpose quite as well, as they are for temporary use and all temporary buildings are abandoned by the contractor or sold for a nominal sum when the work is completed.

Mr. Austin.

Perhaps the greatest obstacle to be overcome will be the lack of discipline. In the average contractor's camp, a large percentage of the laborers employed remain for a short time only, in some camps from 5 to 15 days is a fair average, and many of them are not enthusiastic about shaving and taking baths. They are also not much given to worrying about disciplinary measures, such as being suspended from the payroll or discharged from service. It is evident, however, that much improvement can be made. It will be necessary to arouse more interest on the part of contractors, for, as a rule, they do not take much interest in the sanitary condition of their camps.

If the company letting the contract would insert a clause requiring the contractor to keep his camp in a proper sanitary condition, the engineer in charge would then be in a position to enforce the necessary rules and regulations to bring about the desired result. This would not only improve the efficiency of the service, but would tend to create a favorable impression among the people living in the vicinity of the camp.

Mr. Payrow.

HARRY G. PAYROW, JUN. AM. SOC. C. E. (by letter).—Until recent years, the sanitation of construction camps received little attention by health authorities, due partly to the fact that most camps are remote from populated districts and that the authorities are indifferent. On the other hand, engineers and contractors have not manifested as great an interest as they should, for, as the author states, camp conditions, to a large extent, govern the efficiency of the labor.

The following is a brief description of the methods in actual practice in a construction camp with which the writer was connected. These methods are believed to be simple, of low maintenance cost, and efficacious.

This camp was on the water-shed of a reservoir used as a city water supply, and at that time an additional $1 000 000 dam was under construction. For convenience, the camp was near the work and on slightly sloping ground. An enclosure was formed, with a man-proof fence of 5 strands of barbed wire, and only one gate was provided. Outside of this fence a 2 by 3-ft. trench was excavated all around the enclosure to catch the storm-water, thus preventing it from reaching the reservoir. The trench was graded to carry the water to a sump where it filtered away.

The sleeping quarters consisted of a well-built, one-story wooden building, covered with weather paper. Instead of the old-style bunks, there were comfortable iron beds, a few being on a balcony along one side of the room. A large coal stove was used for heating. The mess-room was a separate, one-story building, and contained long tables with benches and two or three cook-stoves. Connecting the two buildings there was a commissary, where supplies of good quality were

Mr. Payrow.

sold at reasonable prices. All windows and doors were effectively screened.

The sanitary closets were within the camp enclosure, so that the men would not be tempted to go elsewhere. These were ordinary closets, except that galvanized-iron ash-barrels were used as receptacles. These barrels were taken away regularly every day to a farm situated far from the water-shed, and there emptied, washed, sprinkled with lime, and returned the next day. All garbage was also disposed of in this manner, barrels being placed in convenient places for the disposal of waste matter. At the height of fly-time all

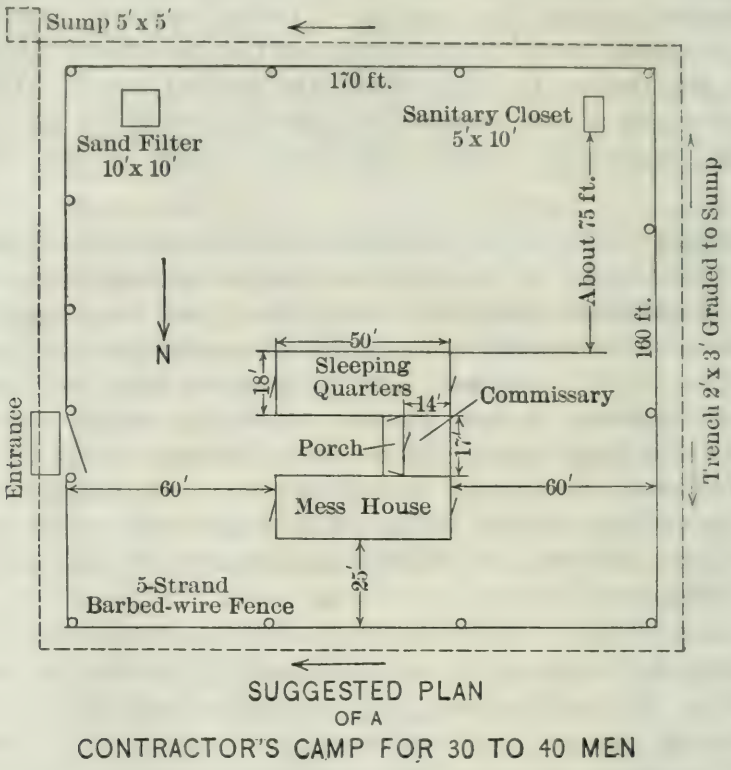


FIG. 6.

sanitaries were sprayed with a formaldehyde solution, which prevented to a large extent the accumulation of flies. For washing purposes, a small sand filter was constructed within the enclosure, and all wash-water was thrown on the filter.

There was a caretaker for the camp, and it was his duty to see that beds were aired daily, floors swept, disinfectant sprinkled about (especially in the sanitary closets), and that the men did not throw refuse carelessly about the enclosure. The patrolman of the water-shed, employed by the city, visited the camp at least once a day and forcibly reminded the caretaker of his responsibility.

The sanitaries on the work were provided with ash-cans which were collected daily and cleaned, as before stated. Any man on the work who was so forgetful as to fail to use the sanitary closet was either laid off or discharged, according to the seriousness of the offense. Mr. Payrow.

Fig. 6, a plan of a contractor's camp, is submitted merely as a suggestion for a layout. Additional sleeping quarters can easily be provided if a larger number of men are to be employed. It is quite essential that there be plenty of yard room surrounding the building, for this tends to keep the men, while idle, within the enclosure.

After the men understood what was required of them, they gladly conformed to the regulations, and very few breaches occurred. They enjoyed the camp, very few of them left the contractor, and there was no trouble in getting men to come on the work. This was in marked contrast to another camp, of the old bunk variety, on the same work. This was conducted by another contractor, and the men were continually disgruntled and constantly leaving.

Harold Farnsworth Gray, Jun. Am. Soc. C. E. (by letter).—The writer is pleased with the kindly reception of this paper by members of the Society. It was his purpose to engage the interest of engineers in the importance of this subject, rather than to write a treatise on camp sanitation. For this reason, brevity was considered essential, and the subject matter was confined to a statement of underlying principles, with illustrations of simple methods of applying these principles to practice. The gentlemen who have kindly contributed to the discussion of this paper have described methods and equipment which, if adapted to local conditions, will work as well as, if not better than, those described by the writer, which were given as illustrations simply because they had been tried out under difficult conditions and had proved effective. Mr. Gray.

An oversight which the writer regrets is that due credit was not given for Figs. 4 and 5. These photographs were kindly furnished by W. B. Herms, Assistant Professor of Applied Parasitology, University of California, in advance of publication of his recent book, "Malaria—Cause and Control."

Mr. Winsor's discussion brings out several important points. His statement, "under all conditions, incineration [of excreta and urine] is much to be preferred to the latrines suggested by the writer," needs qualification. Restating the fundamental principles of waste disposal, the method is in general satisfactory if (1) during collection the wastes are inaccessible to external agencies (*e. g.*, flies), (2) during collection they do not constitute a nuisance, and (3) they are finally disposed of in such a manner as neither to create a nuisance nor to permit infection. Given two or more satisfactory methods of dis-

Mr. Gray.

posal, the decision must rest with the most economical method as regards cost of installation and maintenance.

The minimum limitation of air space in sleeping quarters at 400 cu. ft. per occupant has no real practical significance, in the light of recent investigations. The amount of change of atmosphere is a better criterion for the efficiency of ventilation. For example, a space of 100 cu. ft. per occupant completely displaced four times an hour will in general give better results than a space of 400 cu. ft. changed only once. Small ventilator openings near the ceilings of rooms are cheap and effective. In addition, the men should be encouraged to sleep with windows open.

The writer cannot agree with Mr. Winsor that the enforcement of sanitary regulations is a very difficult matter, unless it is difficult to combine tact with firmness in handling men. He has lived in camps for long periods, in close personal contact with laborers of various types, and has reason to believe that they appreciate a square deal, and that a disposition on the part of the contractor to have some regard for their comfort and health will be met more than half way by the majority. Pride in the cleanliness of the camp can be easily obtained. The illustration given by Mr. Payrow's last paragraph is significant.

Mr. Britton has indicated certain legal phases of the subject. In view of some recent decisions, it is now possible for an employee successfully to sue for damages on account of sickness incurred in camp, if negligence on the part of the contractor can be proved. The imperiling of the health of communities adjacent to camps by reason of unsanitary conditions prevailing therein is a matter within the jurisdiction of local and State health officers, and failure to secure proper conditions in this respect is primarily the neglect of duty by public officials, though the contractor can by no means be excused because of a lax enforcement of the law.

In reply to Mr. Austin's query as to screens, it may be stated that bronze and copper screens do not deteriorate as rapidly as iron, especially in moist climates; they are therefore more economical in the long run, and are to be recommended for all except temporary camps. A rusted and broken screen is worse than useless. Mr. Austin's suggestion as to a clause in contracts requiring the contractor to maintain proper sanitation in camps is excellent, and will work to the benefit of contractor and owner alike.

Mr. Payrow's camp layout, though in general excellent, has one defect. It will invariably be better to separate completely the commissary and mess-house from the sleeping quarters by at least 40 ft.; in winter, a closed passage between the two may be permitted. The use of formaldehyde as described was undoubtedly an extravagance, as it has

Mr. Gray.

no effect on flies, either adults or larvæ, as a contact poison, except in high concentrations and at a prohibitive cost. In warm weather, a few dishes of sweetened 3% formaldehyde exposed where the flies tend to congregate will kill large numbers (the flies drink it eagerly). Pyroligneous acid sprayed about places where flies tend to congregate will be found useful as a deterrent; but, if proper attention is paid to the disposal of wastes (excreta, garbage, manure, etc.), flies, as a rule, will be so few that the use of poisons and deterrents is unnecessary.

Mr. Payrow mentions the use of disinfectants. The distinction between disinfectants, insecticides, and deodorants should be borne in mind, and an article suited to the intended purpose purchased. In buying disinfectants proper, the manufacturer should be required to give (especially if a proprietary compound) a guaranty as to the potency of the product in terms of the U. S. Hygienic Laboratory phenol coefficient, as determined by an analysis of samples by a reputable laboratory. The disinfectant should be purchased on the basis of unit cost (price per gallon divided by the phenol coefficient), selecting, of course, the cheapest in unit cost thus determined. Such disinfectants should be diluted to be equivalent in strength to about a 3% solution of carbolic acid. If proper cleanliness is observed, however, disinfectants will seldom be necessary.

The writer thanks Mr. Miller for his illustration by a concrete example of the value of proper sanitation in camps. He regrets that more such illustrations were not presented, and considers this an indication that proper camp conditions are the exception.

The spirit of Mr. Hardman's criticism is especially fine. The writer freely admits that he is far more interested, as a citizen, in the altruistic phase of the question; but his endeavor was to make this a paper of engineering interest. He intentionally addressed his argument to the pocketbooks of engineers and contractors. He sincerely hopes, however, that, for the honor of the Profession, the members of this Society will be moved in this matter as much by their obligations as citizens as by any economic benefits to be derived.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1254

TUFA CEMENT, AS MANUFACTURED AND USED ON THE LOS ANGELES AQUEDUCT.*

By J. B. Lippincott, M. Am. Soc. C. E.

With Discussion by Messrs. J. M. O'Hara, L. J. Le Conte, Ralph J. Reed, Clifford Richardson, Luther Wagoner, E. D. Knap, O. E. Mogensen, Rapier R. Coghlan, Charles H. Paul, O. W. Peterson, Luigi Luiggi, and J. B. Lippincott.

Los Angeles is situated in a region where the annual rainfall is only 15 in., and where a water supply is requisite, not only for domestic necessities, but also for the beautification of grounds and for all forms of intensive agriculture. The Federal census shows that the population of the city increased from 102 479 in 1900 to 319 198 in 1910, or 211%, which is the greatest growth in any of the larger cities of the United States during this period.

General Description of the Line.—The city relies on the Los Angeles River for its local water supply. This stream rises from the gravel beds of the San Fernando Valley, and is uniform in its flow. It has been completely diverted, and encroachments have begun on the underground waters of the neighborhood. What is known as a miner's inch in Southern California is equivalent to 13 000 gal. per day, or $\frac{1}{50}$ cu. ft. per sec. The right to a miner's inch of water of continuous flow in this locality varies in value from $1 000 to $2 000, and is a measure of the scarcity of local water supplies.

Instead of exercising its right of eminent domain, and attempting to condemn other streams in Southern California, which would result in the depletion of commercially tributary areas, the Board of Water

* Presented at the meeting of December 4th, 1912.

Commissioners adopted the broad policy of going to some distant source for its new supply, where the minimum damage would be done to others, and where the maximum quantity of water would be obtained. As the value of water is increasing very rapidly throughout California, it was decided to construct as large an aqueduct as the city could afford. It was found that the most available supply could be obtained from the Owens River, which, for a distance of 120 miles, drains the eastern face of the Sierra Nevada. There are 40 peaks in this crest, the elevation of which rises above 13 000 ft., Mt. Whitney being the culmination of the range, with an elevation of 14 500 ft. This is the source of the water supply. To the east are the Inyo Mountains, a lower range, blanketed by the high Sierra intervening between the Inyos and the Pacific Ocean, and barren of water crop.

Owens River discharges into Owens Lake, having an area of 64 000 acres, from which the annual evaporation loss is about 6.8 ft. The lake has no outlet and consequently is saline. The aqueduct is being built to deliver a continuous flow of 20 000 miners' inches, or 400 cu. ft. per sec., which is two-thirds of the evaporation loss from the lake. The diversion is made 35 miles above the lake, and the city has purchased 105 000 acres of land in Owens Valley, including both banks of the river from the diversion point to the lake.

The precipitation on the crest of the Sierra Range varies from 40 to 60 in., and is mostly in the form of snow occurring during the winter, the drifts accumulating in the high mountain gorges and melting with the approach of summer. These snowbanks, however, are of such extent that they last over from year to year and some of them have compacted into banks of ice. The high-water period occurs in June and July, low water extending from September through the fall and winter. The annual rainfall in the valley floor is only 5 in.

For the first 60 miles, the capacity of the aqueduct as designed is equal to the mean June flood of the river at the intake, or 900 cu. ft. per sec. This portion of the conduit discharges into the Haiwee Reservoir, which has a capacity of 63 000 acre-ft., with a maximum center height of dam of 91 ft. This Haiwee Reservoir will act as a regulator, from which a continuous flow of 420 cu. ft. per sec. will be drawn. In addition to the surface streams, a large Artesian basin

has been discovered in Owens Valley, from which ground-waters can be drawn extensively during years of drought. In case this supply is insufficient, in the driest years and when the consumption of water approaches the full capacity of the aqueduct, the city can build the Long Valley Reservoir, which site it controls. This will have a capacity of 341 000 acre-ft., with a dam having a maximum height of 160 ft.

For the first 20 miles in Owens Valley the conduit is a dredged earthen canal. This part of the line is in the moist bottom lands of Owens Valley, which are saturated with water, and where the aqueduct will gain water. All other portions of the aqueduct are being lined with concrete. The 40 miles of the aqueduct immediately north of the Haiwee Reservoir, because of its large size, is an uncovered, but lined, canal. With the exception of the first 60 miles previously described, all portions of the aqueduct are lined and covered.

From the diversion point in Owens Valley, the line skirts the eastern base of the Sierra Nevada as far south as the Town of Mojave. It then crosses the western edge of the Mojave Desert, passes under the Coast Range, in a tunnel 26 860 ft. long, and then has three drops aggregating 1 842 ft., where power plants are being installed. Immediately above and below the first two power sites, with drops of 1 516 ft., reservoirs are being constructed.

The capacity of the aqueduct between these two power sites has been increased to 1 000 cu. ft. per sec., in order to provide for the variable power load. With these two reservoirs, it will be possible to increase the flow through the power plants during certain hours of the day to 1 000 cu. ft. per sec., and to regulate it back, below the power plants, to a continuous flow of 400 cu. ft. per sec. In other words, the power factor is taken at 40 per cent. Other large storage reservoirs are being built at the extreme southern end of the line.

The total quantity of cement required for the construction of the aqueduct is estimated at 1 500 000 bbl. Table 1 is a summary of the different classifications of work.

The conduit will deliver water at a point 25 miles north of the city, where the distribution system starts. On March 1st, 1912, 83% of the work was finished, and it is being completed within the estimated time and well within the estimated cost.

FIG. 1.—TYPICAL DESERT CONSTRUCTION, STANDARD SECTION, LOS ANGELES AQUEDUCT. STRUCTURE BURIED; ROOF COVER ALONE IS SEEN.

FIG. 2.—LOS ANGELES AQUEDUCT BETWEEN DIVERSION POINT AND HAIWEE RESERVOIR. DITCH TO DELIVER FLOOD WATER TO RESERVOIR; CAPACITY, 1 000 SEC-FT. CROSSING TO PASS FLOOD WATERS OF COTTONWOOD CREEK OVER CONDUIT, OR DIVERT THEM INTO IT. THE STRUCTURES ARE OF TUFA CEMENT.

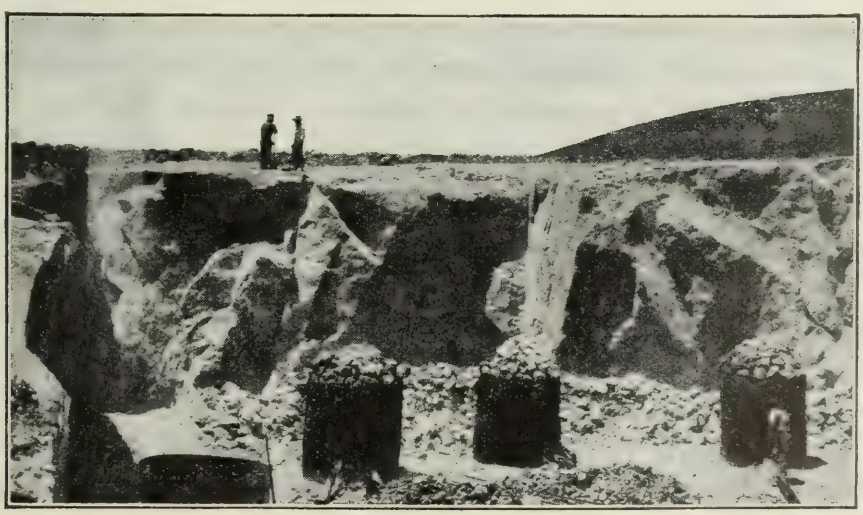


FIG. 3.—FAIRMONT TUFA QUARRY.

TABLE 1.—Lengths and Sections of Various Portions of the Los Angeles Aqueduct.

Classification.	Length, in miles.	Capacity, in second-feet.
Unlined canal	21	900
Open, lined canal	40	900
Haiwee by-pass	2	420
Covered conduit	98	420
Lined tunnels	43	420
Concrete flumes	0.2	420
Concrete pipe, 10 ft. diameter	2.8	420
Steel pipe	9.4	420
Power tunnels	8.8	1 000
Reservoirs	8.5	.....
Total length	233.7	.....

City Cement Plant.—The city has built a standard Portland cement plant on the Southern Pacific Railroad, near the center of the aqueduct line, at a place named Monolith, which is the brand name given to the cement. The mill has a capacity of 1 200 bbl. per day. The operation of the mill is successful, and the cost of producing the cement is reasonably low.

There are six other Portland cement works in California, the products of which are reliable and satisfactory. Apparently, however, there is a definite agreement among these manufacturers as to the selling price. It was not contended that the city could manufacture cement either cheaper or better than some of these larger plants; but the location of the city's cement plant on the line of the aqueduct eliminates 25 or 30 cents per bbl. in freight charges, and it was believed that the city could manufacture its cement on the line of the work at a price which probably would be lower than that for which manufacturers would sell their delivered product. Moreover, by having its own mill, the city is assured of deliveries at the rate required.

The Monolith cement mill is 14 miles from Mojave. A railroad has been built under contract with the city by the Southern Pacific Company, 140 miles long, northward from Mojave, along the line of the aqueduct; because of this special contract, however, freight rates on this new line are high, and amount to nearly 1 cent per barrel-mile.

Three tufa cement-grinding plants have been established on the line of the aqueduct, namely, Haiwee, Fairmont, and Monolith, extensive deposits existing at each of these points. Haiwee, where the northern tufa-grinding plant has been built, is 120 miles by rail

from Monolith, and 106 miles north from Mojave. The southern tufa-grinding plant, at Fairmont, is about 20 miles from the railroad and southwest from Mojave. Transportation charges from Monolith to both Haiwee and Fairmont amount to 90 cents per bbl. As the tufa cement process converts 1 bbl. of standard cement into 2 bbl. of tufa cement, there is saved in transportation charges alone about 45 cents per bbl. on the tufa cement product.

Tuff or tufa is a volcanic, pumiceous rock composed of minute particles bearing indications of having been laid down in water and partly consolidated. Sometimes tufa is a calcareous deposit, but that used for the manufacture of the cement described herein is of volcanic origin. It is of a grayish or creamy color and has a low specific gravity when in rock form; when pulverized, the powder has a specific gravity of 2.2.

According to Dana's "Manual of Geology," puzzuolana is a light-colored tufa found near Rome and elsewhere in Italy, and is used for making hydraulic cement. Puzzuolana is a local name, and tufa or tuff is the geological term. Samples obtained from Italy through the Consular Service in Rome show the puzzuolana to be a light purple-colored fragmental material having somewhat the appearance of volcanic ashes, and unconsolidated. Its analysis is given in Table 2. The tufa used on the Los Angeles aqueduct resembles the German trass, used in the manufacture of the German trass cements.

TABLE 2.—Analysis Record of Various Tufas.

No.	Description.	Date made, 1912.	SiO_2.	R_2O_3.*	Fe_2O_3.	Al_2O_3.	CaO.	MgO.	SO_3.	Loss.	Total.
220	Monolith tufa cement.....	1/15	35.34	11.89	.....	.....	41.05	3.04	.....	8.52	
229	Fairmont tufa, middle of quarry	1/29	71.36	16.15	.....	.....	2.86	trace	0.116	3.22	
230	Fairmont tufa, south side of quarry	1/29	70.06	17.11	.....	.....	1.81	"	0.035	6.51	
233	Monolith tufa.....	2/5	68.26	17.10	.....	.....	2.60	.....	.....	.....	Alkalies.
215	" "	.....	69.46	13.89	2.52	11.37	1.80	2.95	0.429	6.28	4.7
...	Hawaiian lava.....	1/15	51.98	18.75	2.90	15.86	9.57	5.61	.....	.....	
262	Italian tufa.....	3/25	42.36	28.35	.....	.....	9.15	0.54	0.56	13.68	
263	" puzzuolana	3/25	45.68	30.09	.....	.....	11.95	3.76	0.56	6.30	

* $R_2O_3 = Fe_2O_3 + Al_2O_3$.

All the tufa along the aqueduct occurs in beds 100 ft. or more in thickness, and shows distinct lines of stratification, indicating that

it was laid down in water. The beds near Haiwee are in the immediate neighborhood of ancient volcanoes and lava flows. Ancient craters exist in the desert between Fairmont and Monolith. The Fairmont tufa is the purest of the three deposits. It is fine-grained or comminuted, and portions of the bed are entirely free from pebbles or other foreign matter. The Haiwee tufa is quite free from pebbles, but it contains some mica, which makes the grinding slower. The tufa rock at Monolith is more compact, and has a specific gravity of 1.97. It contains a good many pebbles of a flinty character, and occasionally granitic, which makes the grinding slower, but does not affect the quality of the product. The three tufas used on the aqueduct, when blended with cement, are much the same in strength, the average monthly tests showing greater strength first at one mill and then at another.

Silica cement is a term applied to mixtures of cement and silica, usually in the form of sand, ground together in a dry state to a greater fineness than the Portland cement. This mixture is then used with sand and gravel, as in the ordinary method of preparing concrete. The proportion of pure cement is reduced without a corresponding reduction in the strength of the concrete. The voids in the sand are filled with the ground sand, the gradation of the concrete aggregates being thus carried one step farther. This silica is not soluble with lime, and the cement is improved by the regrinding. In 1899 the United States Geological Survey made investigations of sand cement along the upper portion of the Gila River, in Arizona. The tests were made by laboratory grinds and are shown in Table 3. The silica used was a rock known as pearlite, which is a form of rhyolite, from the Butte dam site, and also a quartzite from the San Carlos dam site.

All the tests were made with the same sample of cement. The mixtures were made by weight. It will be noted in Table 3 that, in the case of the silica cements Nos. 1 and 2, all the cement used was passed through a 200-mesh screen. This was unfortunate, because it gave an abnormal fineness, and consequently an undue strength is shown; but, in the case of Nos. 5 and 6, the straight cement was treated in a similar manner, so that comparisons are possible. The sands used were standard in size, but were not standard testing sands.

Comparisons of Test No. 3 with Test No. 5 show the effect of fine grinding on standard cements.

TABLE 3.—RESULTS OF TESTS OF PORTLAND SAND CEMENT.

No.	Material.	Sand.	Portland cement to sand.	Percentage of fineness.			Percentage of water.	Strength, in pounds.	
				50 mesh.	100 mesh.	200 mesh.		7 days.	28 days.
1	Colton and Butte Pearlite, 1 to 1.....	2	1 to 5	0.00	0.00	0.00	10	80	300
2	Colton and San Carlos Quartzite, 1 to 1.....	2	1 to 5	0.00	0.00	0.00	10	90	345
3	Colton regular.....	2	1 to 2	0.42	7.20	33.20	10	170	385
4	Colton regular.....	3	1 to 3	0.42	7.20	33.20	10	140	240
5	Colton fine ground..	2	1 to 2	0.00	0.00	*	12	370	465
6	Colton fine ground..	3	1 to 3	0.00	0.00	*	12	170	260
7	Colton and Butte Pearlite, 1 to 1.....	3	1 to 7	0.00	0.00	0.00	10	75	155
8	Colton and San Carlos Quartzite, 1 to 1.....	3	1 to 7	0.00	0.00	0.00	10	55	185

* Some left on 200-mesh screen.

Since 1903, the writer, from time to time, has investigated the possibility of blending tufas with cement. He identified the Haiwee deposit of tufa, and shipped some of it to the laboratory at the Monolith cement mill. This led to the identification of the ledge of tufa in the immediate neighborhood of the mill and also to the discovery of the Fairmont deposit. The tufa was first ground with cement experimentally in the laboratory, and showed satisfactory tests. The experiment of mixing the ground tufa with slacked lime, without any cement, was also made, and it was found that this would set under water and slowly become hard, but it checked in drying in the pats. This hydraulic property indicated the solubility of the tufa in hydrated lime and its power to combine with the excess lime in the cement. This does not occur with silica cement.

A mill run was then made at Monolith, and a length of several hundred feet of canal was lined with tufa cement concrete, in order to observe its working conditions in the field. As this proved satisfactory, it was decided to build tufa regrinding mills at both Fairmont and Haiwee.

The Fairmont mill consists of a Climax jaw crusher which breaks the tufa to about a 1½-in. size. It is then carried to a No. 8 Krupp ball mill, where it is ground to pass through a 20-mesh screen, or

finer. This ground tufa is then blended in equal parts by volume with the standard cement. This blend is then conveyed to a Gates tube mill, 6 by 16 ft. in size, and the tufa and cement are ground together to a fineness of 90% or more passing through a 200-mesh sieve. The cost of the plant was $27 000. The grinding is much freer during the dry summer months in California than during the wet winter weather. A little moisture in the tufa seriously reduces the product, as it coats the pebbles, thus lowering their grinding efficiency. At the Fairmont mill it was found that, under natural conditions, from 1 200 to 1 500 sacks of tufa cement could be ground per 24 hours. By arranging crude drying devices and driving off the moisture in the tufa with a slow wood fire, this output was increased to from 1 800 to 2 000 sacks per day. In both the Haiwee and Fairmont quarries, the capacity of the ball mill is 40% in excess of the capacity of the tube mill.

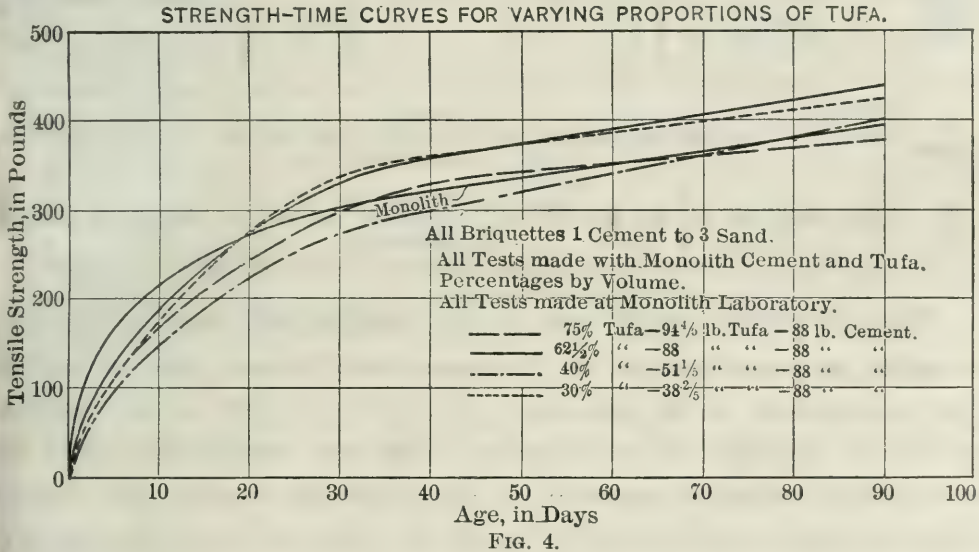


FIG. 4.

Table 2 shows the analysis of various tufas, including the Italian samples and Hawaiian lavas. Tests were made in the laboratory by mixing 4 lb. of tufa, 4 lb. of cement, and 1 lb. of slacked lime. This gave a result from 50 to 75 lb. per sq. in. stronger than when only tufa and cement were used. Subsequently, some mill runs were made with lime added in this manner, but they did not confirm the laboratory tests with lime, and the practice of adding the latter has not been continued. Mill runs were made with varying proportions of tufa at the Fairmont plant, and the results, with different percentages of tufa, are given in Fig. 4. These tests are not fully sustained by later mill and

laboratory tests, in that the higher percentages of tufa at first shown were relatively too strong. Possibly the mill run was not long enough to establish thoroughly the various ratios in the output. Finely ground Monolith tufa mixed with thoroughly hydrated lime in the ratio of 75% of lime to 25% of tufa was made into briquettes with sand in the ratio of 3 to 1. The briquettes were left in a damp closet for 28 days and then immersed in water, and gave the following strengths:

7 days.	28 days.	3 months.	6 months.
40 lb.	80 lb.	70 lb.	225 lb.

The briquettes were softened on the surface by the action of the water.

TABLE 4.—TYPICAL SAND BRIQUETTE TESTS, AT MONOLITH, CAL.

Briquette number.	Date made.	Percentage of water.	Tensile Strength.						Brand.	Percentage of tufa	Fineness.		Setting Time.				Boiling test, 6 hours.
			3 days.	7 days.	28 days.	3 months.	6 months.	1 year.			100 M.	200 M.	Initial.		Final.		
													H.	M.	H.	M.	
20 Tests	1910 Nov.	9	114	203	408	460	518	577	Tufa Monolith Fairmont	50	98	89	2	19	5	49	O. K.
20 Tests	Dec.	9½	104	160	363	460	477	511	Tufa Monolith Haiwee	52	98	91	3	00	6	12	O. K.

Table 4 shows typical tests of the regular mill runs of the tufa cements, as manufactured at Fairmont and Haiwee, and as used in the construction of the aqueduct. A feature to note is the constant growth in strength of the samples. They are occasionally below the standard of strength required by the American Society for Testing Materials for 7 days (from 150 to 200 lb.), but are above the standard for 28 days (from 200 to 300 lb.). As far as tested, the tufa cements manufactured on the aqueduct uniformly show this growth in strength with age, and in this respect are superior to the tests for strength in straight cement, which often show a loss after 28 days. Tests made by the Santa Fé Railway indicate that this loss in strength of straight cement continues, as far as observed, through a 5-year period, in four out of five brands tested.*

As tufa cements are high in silica, and as the silicates of lime are the more enduring but slower portion in the cements, this growing

* *Engineering News*, March 14th, 1912.

strength in tufa cement is quite rational. Straight cements which are slow in hardening show the greatest ultimate strengths, and a high 7-day test is regarded with suspicion.

Briquettes made of pure tufa cement without sand do not show as great strength as neat cement, but as cement is not used in practice in this form, it is relatively unimportant. The leaner the mixture in sand briquettes, the greater the superiority of the tufa cements is shown to be. Broadly speaking, sand briquettes containing 50% of tufa cement show marked superiority in ultimate strength over straight cement sand briquettes.

TABLE 5.—Tufa Sand Briquette Tests, with Varying Proportions of Tufa.

Briquette number.	Date made.	Percentage of water.	Tensile Strength.					Brand.	Percentage of tufa.	Fineness.		Setting Time.				Boiling test, 6 hours.
			3 days.	7 days.	28 days.	3 months.	6 months.			100 M	200 M.	Initial.		Final.		
												H.	M.	H.	M.	
	1911															
21	Nov. 28	9½	85	200	335	370	330	Monolith cement.	55	....	91	2	00	4	20	O. K.
			90	210	345	370	395	Monolith tufa.								
22	Dec. 11	9½	30	100	300	460	390	"	60	....	92	2	10	5	00	"
			35	100	310	435	385									
23	11	9½	35	90	250	350	330	"	70	....	90	2	00	4	00	"
			35	100	260	355	365									
24	11	9½	15	75	210	300	325	"	75	....	90	2	10	5	10	"
			25	80	200	285	375									
25	11	9½	15	75	150	255	300	"	80	....	90	2	50	5	00	"
			20	80	160	235	305									
26	11	9½	15	70	90	140	230	"	85	....	90	3	00	6	10	"
			20	70	90	110	260									

Table 5 shows a series of tufa cement laboratory tests with varying proportions of Monolith tufa with Monolith cement. This table is not in harmony with the mill-run tests shown in Fig. 4, but it appears to be the more rational. It will be noted that there is far less difference in the strengths at the end of 3 months than at the end of 7 days. Unfortunately, the results of the 1-year tests are not yet available.

Table 6 shows the tests of a mill run of 75% Haiwee tufa. It will be noted that the 6-month tests show a fine increase in strength over the 28-day tests.

TABLE 6.—SAND BRIQUETTE TESTS OF 75% MONOLITH-HAIWEE TUFA.

Briquette number.	Date made.	Percentage of water.	Tensile Strength.				Brand.	Percentage of tufa.	Fineness.		Setting Time.				Boiling test, 6 hours.
			3 days.	7 days.	28 days.	3 months.			100 M.	200 M.	Initial.		Final.		
											H.	M.	H.	M.	
15 Tests	1911 Nov.	....	52.8	86	201.6	365	Monolith Haiwee tufa	75	99	91	2	00	5	35	O. K.

Fig. 5 shows graphically the average of all breaks of three tufa cements manufactured by the City of Los Angeles during September, 1911. The standard Monolith cement is blended with 50% of tufa by volume. This is the standard practice on the aqueduct work. In making the test briquettes, the straight cement and tufa cement are mixed with standard sand in the ratio of 3 to 1 by weight, except in the one case shown, where the mix is 3 to 1 by volume. The tufa cement weighs 83 lb. per cu. ft., and the straight cement 95 lb. Standard sand weighs 110 lb. per cu. ft. A mixture by weight, as compared with volume, between tufa cement and straight cement, therefore, gives the former an advantage of 14% in the quantity of cement used. However, in mixing the tufa cement with sand by volume, and straight cement with sand by weight, and making the comparison, this is reversed, as the sand weighs 16% more than straight cement. In making concrete the field practice is to mix by volume, which, in the briquette, gives an idea of the strengths obtained in field practice with the tufa cements. Fig. 5 shows that the tufa cements are slower in getting their strength, usually attaining equal strength with the standard cement in from 6 to 10 days, but continuing to grow in strength, as far as observed, as shown in Fig. 6. All tests made in the aqueduct laboratories indicate this continued growth in strength of tufa cements. The standard cement, however, shows a loss in strength between 1 and 4 months, and then a slow recovery. Other California cement tested in the aqueduct laboratories shows similar loss.

In addition to the laboratory tests of the strength of the tufa, the sands and gravels which are used along the line of the aqueduct, were made into concrete with the tufa cements, and cast into test

slabs, 6 ft. wide and 12 ft. 5½ in. long, and loaded to destruction. These slabs were similar to those used in covering the aqueduct. Table 7 shows the details of these tests. The slabs were reinforced as indicated, and in a manner similar to the reinforcement used in the construction of the aqueduct. The wire mesh used is manufactured by

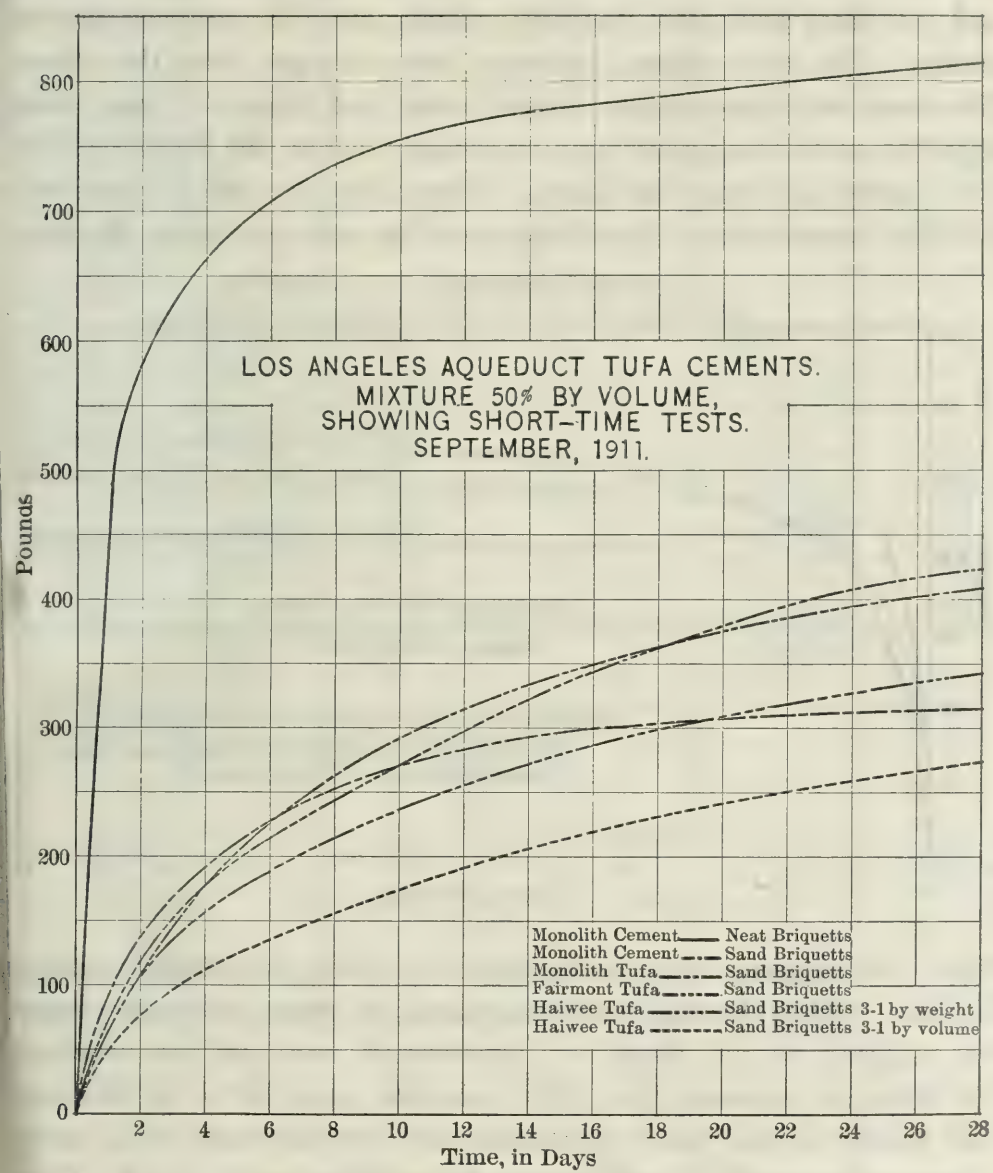


FIG. 5.

the American Steel and Wire Company, being from 58 to 42 in. wide. The practice was to roll these bundles longitudinally along the roof of the aqueduct. The mesh is triangular, 4 in. on a side, with No. 12 longitudinal and No. 14 diagonal wires. The concrete was made by hand, covered with a layer of earth, and kept wet for 20 days, after

which the slabs were dried until they were tested. The slabs were put over piers with a clear span of 11 ft. 5⅜ in. A water load was used, a large canvas bag being put inside of a wooden frame. Tests were first made with earth loads, but the arching effect of the earth destroyed their value. Tufa cements of varying proportions were used, and, in these field tests, the 50% blend gave the most satisfactory results. The tufa cement concretes were stronger than the others. The tests with the straight cement slabs, unfortunately, were made with the reinforcing rods running straight across the bottom of the slab, instead of being bent up at the two sides, as was the case with the tufa cement slabs. In no case were the rods broken in the tests.

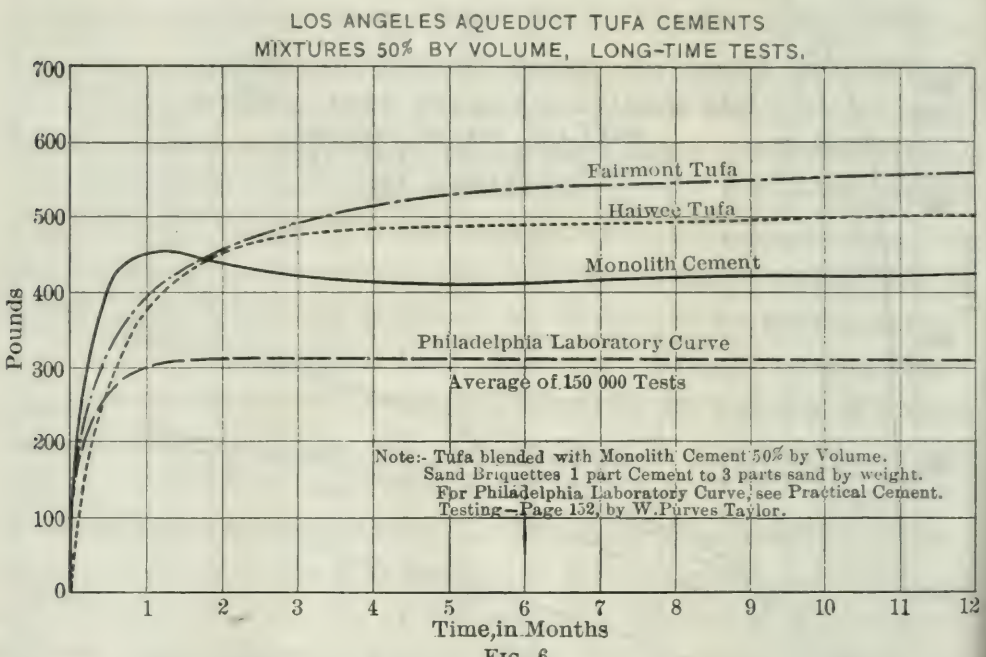


FIG. 6.

The tufa concrete had greater flexibility than the straight cement concrete. After having made this series of tests, the tufa cement was adopted for all classes of construction work on the aqueduct, including the concrete pipe. Five concrete pipes, 10 ft. in diameter, and having a 9-in. shell, reinforced with circular rods 4 in. apart, have now been made of tufa cement, the mixtures being 1:2:4. These pipes have been made for heads up to 75 ft. They have all been filled with water but one, which is necessarily empty until certain other connecting work is completed. Where they have been tested, the pipes are tight, with one exception where a slight circular crack developed. When this pipe was filled with water and soaked up, the crack closed

TABLE 7.—TESTS OF CONCRETE SLABS MADE OF TUFA CEMENT.

Slab No.	Kind of cement.	Proportions of mix.	Date made.	Age.	Description. Dimensions of slabs. Reinforcement: kind, size, how placed.	Manner of loading.	Depth of equivalent earth load at 100 lb. per cu. ft.	Total loads.	Deflections.	Remarks.
3	Tufa-Portland, 50% tufa.	1 : 2½ : 5	2/28	30	12 ft. 5½ in. by 6 ft. 0 in. 8 in. thick at center, 6 in. thick at ends. Four ⅝-in. twisted rods, spaced 18 in. center to center, ½ in. from bottom of slab at center, bent to middle at ends. Wire mesh.	Water load.	0 ft. 8¼ in.	5 000	1/16 in.	Crack at 6 000 lb.
							1 " 4½ "	10 000	¼ "	
							2 " 1 "	15 000	½ "	
							2 " 9 "	20 000	11/16 "	
							3 " 5½ "	25 000	1 "	
							4 " 1¾ "	30 000	1 5/16 "	Poor bearing-R. Broke within 8 in. of rear end.
							4 " 4 "	32 225		
4	Tufa-Portland, 40% tufa.	1 : 2½ : 5	2/24	30	Ditto.	Ditto.	0 " 8¼ "	5 000	⅛ "	Crack at 1 000 lb.
							1 " 4½ "	10 000	7/16 "	
							2 " 1 "	15 000	15/16 "	
							2 " 9 "	20 000	2 1/16 "	
							2 " 10½ "	21 000		Failure. Broke in center. Slow break.
5	Tufa-Portland, 62½% tufa.	1 : 2½ : 5	3/9	30	Ditto.	Ditto.	0 " 8¼ "	5 000	¼ "	
							1 " 4½ "	10 000	7/16 "	
							2 " 1 "	15 000	13/16 "	
							2 " 9 "	20 000	1 5/16 "	
							3 " 4 "	24 000	2 2/16 "	
							3 " 5½ "	25 000		Broke left end. Slow break.
6	Monolith Portland.....	1 : 2½ : 5	2/10	30	12 ft. 5½ in. by 6 ft. 0 in. 6 in. uniform thickness. Four ⅝-in. square twisted rods, ½ in. from bottom of slab. Straight rods. Wire mesh.	Ditto.	0 " 8¼ "	5 000	3/16 "	
							1 " 4½ "	10 000	9/16 "	
							2 " 1 "	15 000	1 7/16 "	Broke 4 ft. from left end. One of the end rods pulled out.
							2 " 3⅛ "	16 500		
1	Tufa-Portland, 30% tufa.	1 : 2½ : 5	2/12	31	12 ft. 5½ in. by 6 ft. 0 in. 8 in. thick at center line. 6 in. thick at ends. Four ⅝-in. square twisted rods, spaced 18 in. center to center, ½ in. from bottom of slab at center, bent to middle at ends. Wire mesh.	Ditto.	0 " 8¼ "	5 000	⅛ "	
							1 " 4½ "	10 000	5/16 "	Crack at 7 500 lb.
							1 " 8¾ "	12 500	⅝ "	
							2 " 1 "	15 000	¾ "	
								15 050		Failed. Rods taken out.
2	Tufa-Portland, 50% tufa.	1 : 2½ : 5	2/17	....	Ditto.	Ditto.	0 " 8¼ "	5 000	1/16 "	Crack in center.
							1 " 4½ "	10 000	3/16 "	
							1 " 8¾ "	12 500	¼ "	Crack ¼ pt. to left.
							2 " 1 "	15 000	⅜ "	Crack at 14 000 quarter to right.
							2 " 5 "	17 500	7/16 "	
							2 " 9 "	20 000	9/16 "	
							3 " 5½ "	25 000	¾ "	
							4 " 1¾ "	30 000	1 "	
							4 " 10 "	35 000	1 5/16 "	No failure. Test dis-

and became tight. In no case have any longitudinal cracks developed, and it is believed that the entire water load is carried by the concrete alone. When concrete takes its set, it shrinks slightly and throws the circular reinforcing steel under compression. Before the steel can carry the load, it must come under tension, and experience with concrete pipe elsewhere indicates that there is enough movement between these two conditions of tension and compression of the steel to cause a longitudinal crack in the concrete unless it carries the entire load. As a practical working test, therefore, this pipe of tufa concrete demonstrates the quality of the material.

The tufa concrete has also been used successfully in the lining of some tunnels in which the ground is exceedingly heavy, and where the sets in the tunnel, made of 10 by 10-in. timbers, spaced from 2 to 3 ft. apart, were repeatedly crushed. The tunnel lining, which has a theoretical thickness of 14 in. between posts, has not shown any failure. In the heaviest ground, however, 6-in. steel I-beams were placed and wedged up against the lagging, the wooden sets then being taken out. This was done because in some instances the timbers were so close together that they reduced seriously the thickness of the concrete tunnel walls. The steel I-beams are left embedded in the concrete as a reinforcement.

Laboratory tests of the tufa product are made continuously at all the tufa mills, and samples are also sent to the main laboratory at the Monolith cement plant. Figs. 5 and 6 represent an average of a month's breaks of briquettes made with equal parts by volume of straight cement and tufa mixed with three parts of standard sand. These are fairly typical of the average monthly mill runs, the tufa cements showing better results than the straight cements and also showing a continued hardening. In the case of the German trass cements, this hardening is known to continue for a period of five years. The lower line on Fig. 6 shows the average of a great number of breaks of various Portland cements at the Philadelphia Testing Laboratories.

A striking feature of the tufa cement is that, in all the four years in which it has been tested, there has never been a pat which failed under the boiling test. This indicates, further, that any free lime which may occur in the cement combines with the silicas in the tufa. Microscopic slides were made of some of the tufa cement and

sand briquettes. They were examined, but no satisfactory conclusion could be reached.

Samples of tufa cement were sent to the Bureau of Standards of the United States Department of Commerce and Labor, and were tested in the Pittsburgh Laboratory. The following quotation is from a letter from this Bureau under date of March 29th, 1911:

"* * * You desired particularly to know whether there was any chemical reaction between the tufa and the cement. The enclosed report shows that such has undoubtedly been the case.

"The addition of tufa or puzzuolana to Portland cement undoubtedly does not reduce the strength of the latter when in the form of a mortar or concrete, but there has always been a question as to whether this is due to purely chemical or chemical and physical phenomena. There has not been any doubt that there is a reaction between the cement and the tufa, but there always has been a doubt as to whether this reaction was sufficient to account for the usual strengths developed by such mixtures.

"The tests were conducted as follows: Mixtures of two parts Riverside cement with one part of tufa, and two parts Riverside cement with one part Ottawa Standard sand, ground 90% through a 200-mesh sieve, were made into briquettes and broken at the end of one-week, four-week, and thirteen-week periods. At the same time, a similar series, using Atlas cement, was carried on, and also the same mixtures used in connection with 1:3 sand briquettes. After breaking the thirteen-week period briquettes, they were dried, the outside surface completely removed, and the interior ground to pass 200 mesh, and used for determinations. By determining the amount of insoluble silica in the cements and in the tufa and quartz, it is possible to calculate the amount of insoluble silica which should be in the dry briquettes (by dry, meaning the complete expulsion of water and CO_2 at 1 000 degrees C.). From the analyses of these residues, there was also obtained the insoluble silica actually present. The difference between these two gives the amount of silica rendered soluble during this period of thirteen weeks. The figures are given in the following table:*

"The following results were obtained in breaking the briquettes:

Riverside.	2 Riverside, 1 tufa.	2 Riverside, 1 quartz.	
0.53	30.97	34.02	SiO_2 insoluble, calculated.
0.53	23.80	32.76	SiO_2 insoluble, determined.
	7.17	1.26	SiO_2 rendered soluble.

* "Tests made at Monolith show 8.4% of Monolith tufa rendered soluble; of the Italian Puzzuolana, 1.6%, and of Italian Tufa, 2 per cent. The presence of this so-called 'soluble silica' is what makes tufas preferable to sands for blending with Portland cements."

Atlas.	2 Atlas, 1 tufa.	2 Atlas, 1 quartz.	
0.59	30.80	33.89	SiO_2 insoluble, calculated.
0.59	24.01	33.00	SiO_2 insoluble, determined.
	6.71	0.89	SiO_2 rendered soluble.

TABLE 8.—Los Angeles Aqueduct Tufa Compared With Silica Cement.

			1 week.	4 weeks.	13 weeks.
Sand.	Riverside cement	1	307	370	416
		2	303	383	413
		3	310	394	390
	Average		307	382	403
	2 Riverside cement, 1 Tufa, 90% through 200 mesh	1	215	373	458
		2	211	302	585
		3	221	341	438
	Average		215	339	457
	2 Riverside cement, 1 Quartz sand, 90% through 200 mesh	1	210	272	286
		2	190	286	286
		3	188	281	303
	Average		196	280	292
	Tufa cement, 50% by volume, Tufa and Riverside cement	1	262	488	534
		2	269	476	561
		3	235	466	478
	Average		255	477	524
Sand.	Atlas cement	1	357	341	454
		2	299	351	484
		3	281	335	469
	Average		312	342	466
	2 Atlas cement, 1 Tufa, 90% through 200 mesh	1	212	350	462
		2	212	398	471
		3	200	410	...
	Average		208	386	466
	2 Atlas cement, 1 Quartz sand, 90% through 200 mesh	1	205	265	314
		2	200	283	362
		3	200	255	367
	Average		203	268	348

The neat briquettes made with tufa cement by the Bureau do not show as much strength as those made with straight cement. This is in harmony with tests made in the aqueduct laboratories. However, when cement is used in concrete, it is always blended with sand, and the sand briquettes tested correspond to practice in construction work. The tests made with the sand briquettes show that the tufa cements have as good strength as the straight cements, or better, and that the blend made of equal parts of tufa and cement is stronger than that made with but 33% of tufa. They also show clearly that the tufa cement, when mixed with sand, gives much better strength than the silica cement, indicating that the tufa combines in a different manner from the silica. In the neat briquettes, however, the silica cements develop the greater strength. The characteristic of the tufa cement continuing to harden substantially with age is indicated in these sand briquette tests. At the time the samples were sent to Pittsburgh, the Fairmont tufa was being blended with Riverside Portland cement. All other tests given of Los Angeles aqueduct tufas were made with the city's cement known as Monolith.

A review of an elaborate series of tests of the German trass cement, is given in a recent engineering periodical.* These are the best available laboratory tests of tufa cements. The investigations were carried on especially to determine the effect of sea water on cements of this class. They were made under the direction of the Prussian Minister of Public Works, and a committee consisting of representatives of the Royal Testing Laboratories, of the cement and trass industries, and Dr. Michaelis, cement specialist. The Royal Testing Laboratories were placed in their charge. Cements of two classes were used, those rich in lime and those poor in lime, and also mortars to which had been added trass and finely ground quartz sand, in order to determine whether trass only acts mechanically by increasing the density of the mortar, or chemically also. The tufas blend slightly better with the cements which are richer in lime. The results of these tests show that the addition of certain puzzuolana (tufa) materials to lean cement mortar is valuable in sea water. The detailed table (Table 9) is given because of the variety in the record,

* *Engineering Record*, August 27th, 1910.

the long period of the tests, and the excellence of the authority. The tests run over a period of five years.*

This cement (Table 9) contained 65.80% of lime, and 23.74% of silica. The cements which have been blended with trass give as much tensile strength as the straight cement when made in sand briquettes, and the samples put in sea water show results of slightly less strength than when they are put in fresh water. The compressive strength of the concrete is less with the trass cements than with the straight cements.

An additional table† shows the strength of mortars and concrete made with a mixture of three parts of trass, two parts of hydrated lime, and one part of sand, giving tensile strengths of 216 lb. in 28 days, 356 lb. in 1 year, and 400 lb. in 5 years. A mixture made of 1½ of trass, 1 of hydrated lime, and 1 of sand gave a strength of 244 lb. in 28 days, 383 lb. in 1 year, and 360 lb. in 5 years.

It is noteworthy in Table 9 that the straight cement sand briquettes, in three out of four instances, show marked loss in tensile strength between 1 and 5 years, the first test alone showing constant strength, while the tufa (trass) combinations show gains in six out of eight cases during the same periods. The two tests showing loss are of samples in sea water.

Dr. W. Michaelis‡ has written an interesting paper on "Portland Cement Reground with Oregon Puzzuolana," in which he enters into a discussion of the chemistry of the problem, and makes a demonstration of the solubility of the tufa with the excess lime of the cement. He shows that puzzuolana (or tufa) will combine with hydrated lime. The series of tests given show the same general results as the tests with German trass cements (Table 9)—that, especially with the leaner mixtures, the tufa blends of equal parts are fully as strong in tension as the straight cement mortars, or superior to them, and markedly better than the "silica cement." In the tests for compression in the leaner concretes (1:3:6), his tufas are as strong; but, with richer mixture (1:2:4), they are about 20% less strong than concrete made of straight cement with the same aggregates.

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* The results have appeared in a report of the Royal Testing Laboratory under the title, "Mitteilungen aus dem Kgl. Materialprüfungsamt."

† *Engineering Record*, August 27th, 1910.

‡ *Cement and Engineering News*, November, 1911.

TABLE 9.—TESTS OF GERMAN TRASS CEMENTS IN SEA WATER (IN POUNDS PER SQUARE INCH).

Age of samples.	Stored in Fresh Water.						Stored in Sea Water.					
	Stored in sand 9 days.			Stored in sand 1 year.			Stored in sand 9 days.			Stored in sand 1 year.		
	Tensile strength.	Compressive strength.		Tensile strength.	Compressive strength.		Tensile strength.	Compressive strength.		Tensile strength.	Compressive strength.	
		Mortar cubes.	Concrete cubes.		Mortar cubes.	Concrete cubes.		Mortar cubes.	Concrete cubes.		Mortar cubes.	Concrete cubes.
1 CEMENT, 2 SAND.												
28 days....	572	6 760	7 040	....	.....	.....	646	6 790	7 170	....	.....	.....
1 year....	722	7 740	7 540	710	8 870	7 500	750	7 470	7 400	....	.....	.....
5 years...	722	6 320	7 590	750	9 530	7 720	612	7 620	7 700	859	11 900	7 820
1 CEMENT, 4 SAND.												
28 days....	348	2 990	3 390	....	.....	.....	362	2 960	3 520	....	.....	.....
1 year....	426	3 400	3 640	524	4 890	4 440	457	3 660	3 960	....	.....	.....
5 years...	362	3 320	4 030	439	3 760	4 340	369	3 440	4 420	442	4 040	4 440
1 CEMENT, 1 TRASS, 4 SAND.												
28 days....	504	3 470	4 290	....	.....	.....	539	3 030	4 670	....	.....	.....
1 year....	616	5 860	5 520	622	5 970	5 820	749	5 900	5 340	....	.....	.....
5 years...	722	6 300	5 570	697	6 040	5 740	583	6 560	5 260	719	6 440	6 070
1½ CEMENT, 1 TRASS, 5 SAND.												
28 days....	573	5 170	4 870	....	.....	.....	597	5 400	4 890	....	.....	.....
1 year....	640	7 200	5 670	617	6 970	6 100	672	7 220	6 120	....	.....	.....
5 years...	724	7 230	6 680	716	6 690	6 090	663	7 570	6 160	744	7 140	6 130
1 CEMENT, 1 TRASS, 8 SAND.												
28 days....	242	1 560	2 120	....	.....	.....	292	1 700	2 140	....	.....	.....
1 year....	353	2 420	2 920	404	3 400	3 240	360	2 690	2 820	....	.....	.....
5 years...	399	2 820	3 230	370	2 370	3 060	394	2 890	3 090	344	2 490	3 120
1½ CEMENT, 1 TRASS, 10 SAND.												
28 days....	240	1 430	2 140	....	.....	.....	262	1 430	2 220	....	.....	.....
1 year....	320	2 240	2 790	347	2 840	3 260	324	2 100	2 760	....	.....	.....
5 years...	329	2 220	3 090	419	3 020	3 330	326	2 240	2 900	497	3 260	2 960

Dr. Michaelis gives the following explanation of the chemical reactions that occur when the tufa combines with the cement:

"These desired hydrates of silica, alumina and iron oxide are found in nature in the form of puzzuolanas, or tufas, volcanic products created by the action of water or steam upon basaltic or granitelike molten formations. They can be artificially obtained by running molten blast furnace slag into water. In both cases the original compounds of the basalt, granite or slag are completely decomposed into their constituents and, furthermore, transformed into comparatively loose material which can easily be crushed. The most valuable part of the various chemical ingredients to be found in a natural or artificial puzzuolana is the silica hydrate, so-called 'soluble' silica which, in distinction from quartz silica, powdered quartz, is soluble in a 10 per cent. solution of sodium carbonate. Such silica combines readily with calcium hydrate and forms an excellent hydraulic cement. To what extent the alumina hydrate and iron hydrate combine with calcium hydrate has not been definitely ascertained. However, from recent researches it appears that especially the alumina hydrate is able to combine with a very large percentage of hydrated lime."

Cost to Manufacture.—The average cost of blending 1 bbl. of straight cement so as to produce 2 bbl. of tufa cement with the small mills installed on the Los Angeles aqueduct is about 74 cents, distributed as follows:

	Cost per barrel of blend.
General expense—labor, live stock, etc.	$0.04
Electric power, at 1.85 cents per kw-hr.	0.105
Quarrying	0.025
Mill operations	0.20
Net milling cost	$0.37

The process of blending 1 bbl. of straight cement with an equal part by volume of tufa gives a resulting product of approximately 10% in volume in excess of 2 bbl. of tufa cement. For this reason, a little more than 1 cu. ft. of tufa cement is put in a sack. A sack of tufa cement weighs 83 lb. The cost of milling tufa cement will vary with the density of the rock. This cost of 37 cents per bbl. of tufa cement applies to all three of the tufa-grinding plants. The tufa at Monolith is denser and slower to grind; but, as this tufa mill is a part of a larger plant, the milling costs are no greater than at the other two places.

Action of Tufa Cement in Field Work.—Tufa cement is more sensitive, and requires greater care in curing, than straight cement, because it is slower in reaching its final hardness. As a rule, Los Angeles aqueduct tufa cements will show as great strength in 7 days as the straight cements, and after that period the tufa cement gains in strength faster than the straight cement. (See Fig. 5.) The tufa cement has to be kept wet longer in hot weather to attain full strength, and is subject to frost longer in cold weather. In slab work, where it is supported by forms, the forms should be left in two or three days longer with tufa cement than with straight cement. When the aqueduct roof slab (which has a span of 11 ft. 5 in.) is made of tufa cement, the forms are stripped in 6 days, in moderate weather. The particular places for which tufa cement is adapted are in massive work, in foundations, and in wet places. It is not claimed that it is suitable for high, thin walls exposed to the dry air of arid regions. It may be, but this has not yet been demonstrated on this work.

Gaugings made in arid America show that the greater portion of irrigation water diverted in earthen canals is lost by seepage before it reaches the fields. A lean tufa cement containing 75% of tufa could be used for earthen canal linings, and would be fully as dense as concrete made with straight cement. It would have sufficient strength to stand up on 1:1 slopes, and it can be given as smooth a surface as ordinary concrete. A length of several hundred feet of open canal lining of this kind has been put in the open flood section of the Los Angeles aqueduct, with 75% tufa. The concrete does not show up as hard in the field as the 50% tufa concrete, but it is satisfactory. Tufa cement, in being more finely ground, adheres somewhat more to the forms than concrete made with straight cement, which is a slight disadvantage. In places along the aqueduct where 50% tufa concrete joins concrete made with straight cement, both being a year old or more, no difference can be detected in the quality of the concrete by picking into it. Plaster made of tufa cement is smoother, and the laborers, after they get used to it, prefer it to straight cement plaster.

Prejudice Against New Cement.—Some cement manufacturers take a stand against tufa cement for two reasons: because it is a cheaper product, with which they would have to compete; and because,

having established a business for a standard Portland cement, anything which might be considered an adulteration would possibly militate against the reputation of all cements. This active opposition has already been encountered among the cement manufacturers in Southern California, who opposed the proposed building ordinance of Los Angeles containing a provision permitting the use of tufa cements. If, however, a product can be furnished which is cheaper in cost and as good in quality, the consumer should have the benefit of it, and undoubtedly will ultimately derive this benefit.

Some foremen and superintendents are also prejudiced against the use of a new product. This is true generally in various branches of industry, and it applies to tufa cement. On the Los Angeles aqueduct, it was found that some of the foremen at first endeavored to avoid the use of tufa cement, but now, after the lapse of two or three years, and having had some practical experience with it, they are willing to accept either that or straight cement from the city mills without any hesitation for all classes of work.

Other Combinations.—Diatomaceous earths are found at various places along the Pacific Coast. Their analysis is similar to the tufas, which they resemble somewhat in appearance and in physical characteristics. A test made with a diatomaceous earth found near Santa Barbara gave the following results:

	Monolith cement. 1 to 3 sand.	Half Monolith and half diatomaceous earth.
3 days	100	210
7 days	200	300
28 days	310	370
3 months	330	620

The briquettes were made as provided for in specifications for testing cements. Only one set of tests was made, and this table is not given as conclusive evidence.

In the Hawaiian Islands, the volcanic rocks prevail. As far as the writer's knowledge of that country extends, there are no lime deposits suitable for the manufacture of cement. The black basaltic lava has been analyzed by the branch of the Department of Agriculture located on the islands, and the following contents determined:

SiO_2	TiO_2	Fe_2O_3	Mn_3O_4	K_2O	Na_2O	CaO	MgO	FeO	Al_2O_3
51.98	1.50	2.90	0.92	0.97	2.70	9.57	5.61	6.84	15.85

It is difficult to grind this lava in the mills. There is another form of volcanic material, locally called red clinker, which resembles somewhat in appearance a cement clinker. Experiments were made in grinding with Santa Cruz cement both the basaltic lava and the red clinker in a local tube mill. The results are given in Table 10. After making the first three tests, 1½% of gypsum was added, which improved the strength of the material. All these tests were made with mixtures of one part of the cement indicated to three parts of standard sand. While the results did not seem to be as satisfactory as those obtained with the Southern California tufas, nevertheless, enough strength was developed to indicate the possibility of blending these lavas with cement in such a way as to result in an economy.

It will be noted that the chemical analysis of the lava quite closely resembles the analysis of tufa, and it is this resemblance of chemical properties which suggested the experiments with the lavas. There is a marked difference in their physical characteristics as compared with the tufas. They have not been comminuted by contact with water as the tufas have, a process which is considered important, if not essential.

Conclusions.—The following conclusions are drawn for tufa or puzzuolana cements:

1st.—The tufa, when finely ground with cement and used in concrete, combines both chemically and mechanically. Blends of 50%, when mixed with sand, give greater tensile strength after 10 days than straight cement mixed with the same proportion of sand. The leaner the mixture, the greater the relative superiority of the tufa cement. In compression, the tufa cement concrete is less strong (20%) in rich mixtures (1:2:4), and as strong in leaner mixtures (1:3:6).

2d.—Tufa cements, in tension, of blends from 30 to 80% show a continued growth in strength with age, as far as tested, up to 5 years, and in this respect are superior to straight cements, which usually show declining strengths.

3d.—The tufa concretes must be handled with greater care with reference to both cold and drying, and forms should be left in place about one-third longer. In massive work this feature is negligible.

4th.—From the fact that the tufa cement is more finely ground and, in part, combines mechanically with other aggregates, carrying

the gradation of fineness one step farther, it makes a denser and more impervious concrete.

5th.—Where cements are high priced, a substantial economy may be effected if deposits of tufa are available. These conditions occur in portions of Western America. The milling cost of producing the extra barrel of tufa cement in small plants should not exceed 75 cents.

TABLE 10.—Sand Briquette Tests Made with Hawaiian Lavas.

Briquette number.	Date made.	Percentage of water.	Tensile Strength. 3 days.	7 days.	28 days.	3 months.	Brand.	Percentage of tufa.	Fineness. 100 M.	200 M.	Setting Time. Initial H.	Initial M.	Final H.	Final M.	Boiling test, 6 hours.
a	1911 Nov.	10½	145 135	250 260	335 345	435 465	Santa Cruz cement and sand	....	93.8	.....	2	15	6	30	O. K.
b	"	10½	55 105	165 175	250 245	320 330	Red clinker and sand.	50%	.....	91	0	40	1	25	"
c	"	10½	110 75	140 150	185 195	250 275	Lava basalt.....	50%	.....	92	0	40	1	20	"
27	Dec.	10½	35 35	100 110	145 150	....	Red clinker Santa Cruz.....	50%	.....	91.4	0	40	1	25	"
28	"	10½	125 130	200 210	325 355	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
29	"	10½	100 110	200 200	340 350	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
30	"	10½	125 130	190 200	330 335	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
31	"	10½	145 145	190 200	310 325	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
32	"	10½	120 120	215 230	350 340	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
33	"	10½	135 140	200 200	320 365	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
34	"	10½	120 140	195 200	350 345	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
35	1912 Jan.	....	145 140	245 255	330 330	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
36	"	....	130 140	220 225	300 310	....	Red clinker, 1½% gypsum added...	50%	.....	.....	2	30	6	00	"
37	"	10½	55 55	120 125	180 190	....	Blue lava.....	50%	.....	.....	1	00	11	00	"
38	"	10½	80 80	135 150	275 275	....	"	50%	.....	.....	1	00	11	00	"
39	"	10½	60 65	150 160	240 255	....	"	50%	.....	.....	1	00	11	00	"

The development of the tufa cement on the aqueduct, as is usually the case with affairs of this kind, is the result of the co-operation of a number of different parties. Mr. E. Duryee, Cement Chemist for

the aqueduct at that time, conducted the preliminary experiments. Mr. G. M. Andrus, who succeeded Mr. Duryee as Cement Chemist, has done a great deal in investigating these cements. Mr. Andrus has been of much assistance to the writer in the preparation of this paper. The cement plant is under the direction of Mr. Roderick MacKay, Mechanical Constructor, and William Mulholland, M. Am. Soc. C. E., is Chief Engineer in general charge of the work on the aqueduct. The writer has been Assistant Chief Engineer since the beginning of the work.

DISCUSSION

Mr. O'Hara.

J. M. O'HARA, ASSOC. M. AM. SOC. C. E. (by letter).—According to Professor Eakle, of the University of California, the rock ground with the cement used on the Los Angeles Aqueduct, is a rhyolite-tuff and a trachyte-tuff, which, when finely ground, will possess the same characteristics as finely ground clay.

The cement produced by the method used at Monolith and Haiwee, Cal., is similar to cement adulterated with clay. Rhyolite-tuff and trachyte-tuff are not to be confounded with the volcanic rocks known as puzzuolana and trass, which have been used for the manufacture of cement. Puzzuolana and trass are the hardened products of volcanic action in their original state, in which respect they are similar to blast-furnace slag. On the other hand, the volcanic tuff of the nature found at Monolith and Haiwee, is not comparable to blast-furnace slag, being more of the nature of altered volcanic rock.

In Professional Paper No. 28, issued by the United States Government, through the Corps of Engineers, it is stated:

"Puzzolan cement never becomes extremely hard like Portland, but Puzzolan mortars and concretes are tougher or less brittle than Portland. * * * It is unfit for use when subjected to mechanical wear, attrition, or blows. It should never be used where it may be exposed for long periods to dry air, even after it is well set. It will turn white and disintegrate, due to the oxidation of its sulphides at the surface under such exposure."

A series of tests made at the Ohio State University, under the direction of Professor C. E. Sherman, seems to prove that clay, up to 15% of the weight of the sand, adds strength to the mortar. One plausible explanation for this increased strength is that the addition of smaller particles of material aids in filling the voids in the sand. One detrimental result was definitely established, namely, that mortars made from sand carrying clay could not be placed under water safely, because the clay softened and warped under its influence.

Mr. Lippincott states:

"As tufa cements are high in silica, and as the silicates of lime are the more enduring but slower portion in the cements, this growing strength in tufa cement is quite rational. Straight cements which are slow in hardening show the greatest ultimate strengths, and a high 7-day test is regarded with suspicion."

Tufa and Portland cement, as used on the Los Angeles Aqueduct, is a mechanical mixture, the two materials being blended in equal parts by volume. Under these conditions, no silicates of lime are found, and the gradual increase in strength is not due to the same cause as the slow hardening of a high silica Portland cement.

In setting under water, especially sea water, the lime set free from cement by the action of the water combines with soluble silicates, and maintains the strength of the mass. The percentage of lime set free is small, however, and in the Aqueduct product would not combine with much of the silica. When the concrete sets in air, little or no combination with the tufa will take place, and it is only present as an adulterant. Mr. O'Hara.

In the manufacture of Portland cement to-day, the clinker is ground much finer than was the custom some years ago, and the lime factor is higher. Consequently, the calcium silicates hydrate more readily, and ultimate tensile strength is reached at an earlier period. Besides, the flash strength, due to the calcium aluminates or gypsum, should not be mistaken for the final strength, due to the calcium silicates.

A high 7-day strength is not of itself an indication of poor quality in a cement, all other things being normal; it may be an indication of early ultimate strength.

The writer is of the opinion that the trend of the paper, to show that the tufa cement, as made at the Los Angeles Aqueduct, is as good as a Portland cement, is dangerous and without precedent.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The natural property possessed by tufa of combining with the free lime in all Portland cements, is certainly a very great discovery, the full importance of which cannot be over-estimated. This interesting feature is further emphasized by the announcement of Dr. Michaelis that his tests, extending over five years, show conclusively that the addition of tufa to lean Portland cement mortar is valuable in sea water, this latter being especially important to harbor engineers. In view of the general experience that Portland cement concrete slowly disintegrates in sea water, Italian harbor engineers usually recommend a mixture of puzzuolana and Portland cement for all concretes exposed to its ravages. In many cases where unit strength and quick setting are uncalled for, they use a mixture of ordinary fat lime and puzzuolana. This shows a tendency to return to the old Roman practice, long before the time of Vitruvius, 14 B. C., and recalls the old adage: "Verily, the footprints of the old Roman engineers are eternal." Mr. Le Conte.

The writer's attention was first called to the fact of the falling off in strength of Portland cement briquettes after 1 year, while on the fortifications at and near Fort Point, San Francisco, Cal., in 1895 to 1897. At that time some 350 000 bbl. of European Portland cements had been used. The records showed a general falling off in strength after 1 year, a few samples extending to 10 years. The same feature was noticed in the reports of the Metropolitan Water Board, Boston, Mass., and seems to be the experience everywhere.

The author's experiments, indicating that sand briquettes con-

Mr. Le Conte.

taining 50% of tufa cement showed marked superiority in ultimate strength over those with straight Portland cement after 6 months, are certainly most encouraging.

Years ago, the writer decided that the well-known "Dyckerhoff" brand of Portland cement was the best in the market, but its use was prohibited by its high cost due to transportation expenses from the works to the seaport. In private works under his charge, he overcame this difficulty with a 1:5:10 concrete, which gave highly satisfactory results. It would be exceedingly interesting to know how tufa cement in the same proportions would behave.

There seems to be a fad for demanding "sharp coarse sand" in specifications for concrete. This fad is based on the general craze for short-time high tests, and nothing else. Engineers often go to great expense to get good coarse sand to make a high-test record, when, in point of fact, after 3 months, briquettes made with fine beach sand show just as great strength as those made with coarse sand, if not greater; that is, after the expiration of 3 months, all the advantages of using coarse sands have entirely disappeared. The cost of manufacture given by the author is most encouraging, and puts new life into the industry.

The lining of irrigation canals in leaky ground is certainly a most serious question, and all sorts of expedients are being tried to overcome the practical difficulties. The proposed lining with lean concrete seems to be the best solution of the problem. The experiments with Kieselguber are very instructive, and broaden the entire field of investigation.

Mr. Reed.

Ralph J. Reed, Jun. Am. Soc. C. E. (by letter).—It is obvious that, on account of finer grinding and consequently better mechanical combinations with other aggregates, tufa cement must make a more dense and impervious concrete. It would have been interesting to have presented some experimental data showing comparisons between the permeability of straight Portland cement concretes and those made with tufa Portland cement. Doubtless experiments along these lines have been carried on in the Los Angeles aqueduct laboratories.

The writer wishes to bear witness to the excellent appearance in general of the concrete work on the aqueduct. A large portion of this work has been constructed with tufa cement concrete. By far the largest portion of the work has been completed under conditions which those at all familiar with Western deserts will recognize as far from ideal for the most successful concrete work. It has apparently been difficult at many points to obtain first-class aggregates. Water for sprinkling the finished work has been hard to supply, and, in fact, the many difficulties encountered are appreciated only by those connected most closely with the work. The writer has been especially impressed with the freedom from cracks shown by the finished tufa

cement concrete work in tunnels, open and covered conduits, and especially in the large reinforced concrete siphons referred to by Mr. Lippincott. Mr. Reed.

That the tufa cement must be handled with greater care during drying is apparent from the fact that in the vicinity of open manholes along the covered conduit in the Mohave Desert, where the concrete during the later stages of drying has been exposed to the sun, cracks are frequent. A little farther back, where the atmosphere has been humid on account of water left standing in the conduit, very few cracks are noticeable. In the tunnels, where the atmospheric conditions, and especially humidity, have been most completely in control, the tufa concrete lining is dense, hard, and tough, and rarely shows any cracks.

Clifford Richardson, M. Am. Soc. C. E. (by letter).—The title of this paper might well have been puzzolan cement, instead of tufa cement, in order to make it more intelligible to the cement expert. From a geological point of view, tufa is, no doubt, a more accurate designation of the material in use in California than puzzolan, but the latter is commonly used, as witnessed by the title of Part VII of Eckel's "Cements, Limes and Plasters," which is "Puzzolan Cements." Eckel includes among puzzolanic materials "all those natural or artificial materials which are capable of forming hydraulic cements on being simply mixed with lime, without the use of heat." It is stated by Mr. Lippincott that by mixing the ground California tufa with lime, without any cement, it would set under water and slowly become hard. The material, therefore, may be recognized as being puzzolanic. From the analyses of the tufa in use in California, it appears that those denominated Fairmont and Monolith contain a much higher percentage of silica than is found in the puzzolans from Italy and France, and in the trass from Germany, the percentage of silica in the Italian materials varying from 39 to a maximum of 63, in those of France from 31 to 48, and, in the case of the trass from Germany, 46 to 67.6, according to Eckel. The California material corresponds more closely in composition to santorin earth, which carries as much as 73%, and averages 66.4%, of silica. Unfortunately, no data are available showing what proportion of the silica in any of the materials occurring abroad, or in those in California, is soluble in acid, as it is hydrated silica of this description which possesses the greatest hydraulic value. There seems to be no question, however, but that the ground mixture of Portland cement and California tufa produces a true puzzolanic cement. Mr. Richardson.

It appears that the tufa in California is ground and then blended with the Portland cement. It is natural to suppose that the tufa cannot be economically ground unless it has been thoroughly dried, in view of the general experience in the cement industry with grinding

Mr. Richardson.

slags and other raw materials. To the writer it would seem that it would have been better practice to grind the tufa and the Portland cement clinker together at the same time, in the same mill, as a more intimate mixture of the materials would result.

One fails to find in the paper the results of any tests of the tufa cement with the local sands in use, either in tension or compression. Data of that description would be instructive, especially as it has been recognized, at least in Germany, that tensile strength tests of cement are of small importance as compared with those in compression, and, on this account, have been abandoned there.

Mr. Lippincott states that:

"A striking feature of the tufa cement is that, in all the four years in which it has been tested, there has never been a pat which failed under the boiling test. This indicates, further, that any free lime which may occur in the cement combines with the silicas in the tufa."

To the writer, this does not seem to be a conclusion which can be justified. It merely means that the cement has been ground finer by passing it through the Gates tube mill after the two materials have been blended, and, consequently, that it does not contain the coarse particles which cause pats to fail under the boiling test, a test which the best German authorities consider of very little value, and which seems to the writer to bear no relation to the conditions to which the cement is exposed in actual construction.

It appears that, in its Pittsburgh laboratory, the U. S. Bureau of Standards has determined the amount of insoluble silica in the cements and in the tufa, but, unfortunately, the percentages of soluble silica are not stated in the paper, although there is sufficient evidence that there is a considerable percentage of silica in the tufa in a hydrated condition which, in the cement in a set condition, is in the soluble form, although there is no evidence that it is combined with lime.

It would be interesting to determine in some manner the hardness of the puzzolan cement in concrete and its resistance to attrition and impact, as compared with Portland cement. It is frequently deficient in this respect, and is not always as satisfactory as concrete made with Portland cement. Mr. Lippincott is quite right in stating that a cement of this type is more satisfactory when used in moist places and in masses where it cannot dry out, than where the latter is possible, and, on this account, work done with it requires greater care in order to permit of satisfactory setting than that done with straight cement.

Based on the writer's experience with sand cement, it would seem that there was plenty of justification for using tufa cement in the California work, but it will be interesting to watch its behavior during a long period of years.

Mr. Wagoner.

LUTHER WAGONER, M. AM. SOC. C. E.—Influenced by the reports of the successful use of tufa by the Los Angeles Aqueduct Board, the speaker located several tufa claims for a contemplated work in Arizona, and until recently supposed that the successful use of tufa was definitely settled. In 1911, some parties desired to build a dam and other works near Prescott, Ariz., which would require large quantities of cement. The speaker furnished them with samples of his tufa and was informed that they had also used these and other near-by samples in combination with Portland cement from Iola, Kans., and Portland, Colo., and that all the samples thus tested gave poor or negative results.

On the speaker's return homeward, he compared samples of the Arizona tufa with those of California tufa, in the Aqueduct office at Los Angeles, the samples appearing to be nearly the same in physical characteristics. Both seemed to be rhyolite tufa.

In June, 1912, he received a copy of a "Report on Municipally Manufactured Cements, Los Angeles Aqueduct," signed "Gervaise Purcell, W. H. Sanders, F. C. Finkle, Chester B. Loomis, Consulting Engineers." This was a report made to Mr. Edward M. Hagar, President of the Association of American Portland Cement Manufacturers, Philadelphia, Pa., in which they attacked vigorously the results obtained by mixing tufa with the cement made by the Aqueduct engineers. They claim that the hydraulic index of the municipally manufactured cement is as low as 1.66 to 1.69, while average American Portland cement has an index averaging 1.91, and German brands 1.98; and they also claim that the tufa used is the physical equivalent of clay, hence the poor results found by them.

Thus far, the speaker has not heard of any public denial by the Aqueduct Board of the truth of the above quoted charges, and the whole matter is thus enveloped in doubt. It would certainly be a matter of considerable interest to the Profession to have careful tests made of the said California tufa, using a standard brand of cement, so that this element of uncertainty as to hydraulic index might be removed, and at the same time settle the question, whether the real benefit is not due so much to the character of the adulterant as to the fact that the regrinding with the tufa brings a larger percentage of the otherwise inert, non-setting grains of Portland cement into a finer state, whereby, when wetted, they "gel" or enter into a colloidal condition.

Concerning the incidental matter of fine grinding, the speaker at one time made tests on a certain fine-grained ore carrying silver chloride, the pan tailings assayed 12 oz. per ton, and 11% of the silver was soluble in "hypo"; 1 gramme of the tailings was ground for 1 hour in an agate mortar with "hypo", and every effort was used to crack and break up the grains of mineral, the result being that 50% of the silver was made soluble. On washing and separating the

Mr. Wagoner. coarsest grains, the latter were found to have a maximum diameter of $\frac{1}{100}$ mm. This test shows that the finer grains probably have a cushioning effect which protects the larger ones from effective action, and this raises the question: Could not the present methods of grinding clinker be greatly improved by using a gradual reduction method in which the fine dust would be removed more promptly, and thus enable much finer grinding of the cement, and probably with greatly improved efficiency, by enabling a larger percentage to act as colloids?

With his present information on the matter, the speaker considers that there are still several points to be cleared up before venturing on the extensive use of tufa as an adulterant or adjunct to Portland cement.

Mr. Knap. E. D. KNAP, Assoc. M. AM. SOC. C. E.—The speaker has looked up some records relative to one phase of this tufa cement question, that is, of Portland cement ground with varying percentages of silica, which makes a compound not unlike that of tufa, in that the main ingredient of the latter seems to be a form of silica. These records show the results of tests up to 6 months for mixtures in which the Portland cement ranges from 100% down to 12½ per cent. The strength of the 100% mixture starts with 417 lb. at 7 days, and runs up to 650 lb. A Portland cement ground with an equal weight of white quartz, making a 50% cement, starts at 270 lb. at 7 days, and runs up to 653 lb. at 6 months, which is 201% of the strength of the neat cement, considering the cement content of the mixture.

There is another compound which consists of the second, that is, the 50% Portland cement, and 50% sand, one part each of sand and cement ground together, and another part of crushed quartz mixed with it mechanically. This compound started at 257 lb. and ran up to 614 lb. in 6 months, which is 283%, counting the Portland cement at 33⅓ per cent.

There was still another mixture which was one part each of cement and sand ground together, with two parts of quartz added, making a 25% mixture. This mixture started at 264 lb. and went to 585 lb., which is 360%, considering the quantity of cement it contained.

From these figures, it would seem that the chief thing to be gained by adding and grinding together silica of any kind and Portland cement, is first to provide a filler, and secondly to increase the impalpable portion of the Portland cement, because everybody knows that only a small part of the cement in the average Portland cement is of any use, and that is the impalpable powder. The remainder acts as a filling to hold it together, and by grinding quartz and cement together, the binding proportion is very much increased by the additional fine grinding, which cannot be obtained in any other way, at least, it cannot be obtained commercially in any other way.

Mr. Knap.

For instance, in addition to this 25% mixture, there is another 25% mixture of the same brand of Portland cement mixed with three parts of standard crushed quartz sand, as in ordinary mortar, and that starts at 134 lb., as against 264 lb., and only runs to 388 lb. at 6 months, which is very much less than 585 lb.; and it seems to the speaker that this "diluted" cement, this tufa, is beneficial chiefly in that, for a certain quantity of cement used, a very much greater strength is obtained because of this grinding, as the fine content fills the voids. For instance, the strength of the 25% mixture of sand and cement in the ordinary way as mortar, and the strength of the corresponding mixture partly ground together, is as 1 to 1½; that is, the mixture ground together is 151% of the strength of the ordinary mortar; therefore, it would seem that, aside from the possible combination of some of the free lime, or the extra lime, with the soluble silica of the tufa, as against the insoluble silica of the quartz, probably the greatest advantage is derived from the extra grinding.

The speaker has also the results obtained with a 12½% mixture, a mortar made with three parts of standard crushed quartz sand and one part of the 50% "ground together" mixture previously mentioned, which, at 6 months, shows a strength of 126%, considering the cement content. This mixture, however, starts at only 96 lb. at 7 days and takes 28 days to reach the minimum strength allowed for 7 days. Obviously, this would unfit it for use in any work requiring early strength, no matter how strong it might become later; and this same trouble, namely, the lengthening of the time required for work to "stand alone," seems to be one of the strongest arguments against "diluted" cements in general. They all seem to be good enough in the course of time, but time is often a governing factor. This is true, not only in the case of form work, where every day that the forms have to be left in place causes additional expense, but even more so in cases where frost or bad weather must be considered.

The time consideration applies somewhat to the remarks of others in reference to fine *versus* coarse sand. A sand which will give great ultimate strength may be unfit for use in many cases, because immediate strength also is required. The speaker's experience with different sands, however, leads to the conclusion that, to say that a fine sand is poor and a coarse sand is good is not in all cases accurate. He has found good fine sands and poor coarse ones. The measure of efficiency in any sand, it would seem, is in the difference in the relative sizes of the grains composing it, rather than in the actual maximum size of some of the grains; or, expressed differently, in the absence of voids, which are likely to be much less in a sand having grains with a great range of sizes than in one in which the sizes are more uniform.

Mr. Mogensen

O. E. Mogensen, M. Am. Soc. C. E.—During the past two years more or less comprehensive articles have appeared in the technical periodicals mentioning this half-and-half mixture of cement and tufa used on an extensive scale on the Pacific Coast. The subject is an important one, of interest to engineers throughout the United States, and the speaker, therefore, was pleased to see it described so thoroughly and comprehensively before this Society.

The high results in tensile strength and the generally good behavior of the laboratory tests, described so fully in this interesting paper, are obtained partly by the chemical reaction and partly by the process of re-grinding in the tube-mills the straight Portland cement mixed with tufa to a fineness greater than that generally required for Portland cement. By this process the two materials are mechanically and intimately mixed and pulverized, which, of course, could not be done on the mixing board. This applies to tufa, or puzzolan, or to slag.

The question as to why fine sand, which is generally known to produce an inferior mortar as compared with coarse sand, in this instance gives such high tensile strength, is readily explained, and is really the secret of the whole process; it is due to the re-grinding and intimate mixing process in the grinding mills.

The combination of Portland cement with a certain percentage of puzzolan is a subject which occupies many minds at the present day; and, whether it be as tufa cement, slag cement, sand cement, or cement mixed with diatomaceous earth, the speaker agrees with the author in his statement: That the consumer should derive the benefit, if a product can be furnished which is cheaper in cost and as good in quality. To this the speaker would add: and provides the same degree of safety to structures occupied by people, or to structures otherwise dangerous if insecure. It must be borne in mind that a large percentage of the cement manufactured, perhaps the largest, is used for concrete in bulk, and for work of lesser importance, and for this a puzzolan cement will meet all requirements, and in certain cases be more satisfactory than straight Portland.

It is natural that the cement manufacturers on the Pacific Coast should oppose such a radical step as this, but it is certainly a very short-sighted policy when the finished product, the concrete, as claimed in the paper, has been proved to be free from defects, and continues to give good results.

Table 1 is a summary of the different classifications of the work. It is not clear from this table that the tufa cement was used in the construction of the 91-ft. dam of the Haiwee Reservoir. The paper describes its application for the lining of canals, the covering of conduits, and for pipes, tunnels, and reservoirs. The speaker would like to know if the engineers of the Los Angeles Aqueduct made any

discrimination against the use of the tufa cement by excluding it from structures in which possible defects would have serious consequences, causing loss of life. Such discrimination would seem very natural, as no one can predict, even from the most elaborate and carefully conducted laboratory tests, how a new material, such as concrete made from this tufa cement, will stand in the course of years. Mr. Mogensen.

Mr. Wagoner's personal opinion and expressions of the views held by some of the engineers on the Pacific Coast are not very encouraging, although not convincing, as they reflect a doubt as to the stability of the work. It is to be hoped that several disinterested engineers who have given this work time and study will contribute liberally to the discussion of this paper. If the author's conclusions, which apparently agree with the results obtained in Europe with like materials, the so-called trass cement, are borne out by the test of time, the knowledge of this material and process cannot be spread too soon for the benefit of the Engineering Profession.

There are, of course, sections of the country and many cement plants where proper materials of this and of similar composition are not available or cannot be obtained at a reasonable price, where no benefit would be derived by deviating from the standard Portland cement practice. This fact, however, should not mitigate against those plants and those communities which would be in a position to benefit by the combination referred to.

Under the heading, "Other Combinations", the author mentions that diatomaceous earth resembles the tufas in their analyses and in physical characteristics, and he gives the result of a set of tensile tests of one-half Monolith cement mixed with one-half diatomaceous earth. At the Congress of the International Association for Testing Materials, New York, 1912, a paper on this subject entitled "Diatom-Earth as Puzzolan for Cement," was presented by Mr. A. Poulsen, Chief Engineer, to the Danish Government Maritime Works. In this paper Mr. Poulsen describes the favorable results he has obtained by mixing diatomaceous earth, in proportions determined by chemical analyses, with Portland cement clinker, the two substances being pulverized together in the mill. The cement thus made has shown remarkably good results by producing a concrete which has the property of resisting very effectively the action of sea water. Mr. Poulsen's theory agrees with the author's, in so far as the laboratory tests he has conducted show that a chemical reaction takes place between the cement and the diatomaceous earth. The free lime, which he estimates constitutes 30% of the weight of Portland cement, and which in sea water becomes the direct source of disintegrating the concrete by being transformed into calcium sulphate, combines

Mr. Mogensen.

with the silica hydrate and the alumina hydrate in the diatomaceous earth. The diatomaceous earth which he used contains:

Soluble silicic acid (SiO_2).....	51.6 per cent.
Soluble alumina (Al_2O_3).....	8.5 " "
Making a total of.....	60.1 per cent.

of active substance available for the absorption of the free lime in the concrete.

Mr. Poulsen's paper also contains the results of tests of the tensile strength, of the chemical relations, and the compressive strength of mortars with and without puzzolan. By these tests he wishes to point out the distinct advantage of using diatomaceous earth, as compared with the trass or tufa, and the results agree essentially with those of the author in respect to the advantage of the leaner sand mixture as compared with the richer mixtures.

Mr. Poulsen's statement that the cement when mixed with water contains free lime to the extent of 30% of the weight of Portland cement surprised the speaker, but, in discussing this matter with his associate, Mr. Pontoppidan, Chemical Engineer, the latter confirms the presence of a considerable amount of free lime in mortars made from Portland cement, and explains the chemical reaction which takes place in the following manner:

"This 'free' lime, which is Ca $(OH)_2$, calcium-hydrate, or what is ordinarily called slaked lime, crystallizes as such in long needles, and is important in giving the cohesive strength to the mortar. When Portland cement is mixed with tufa, trass, or 'Moler' (diatomaceous earth), and then made into a mortar with water, the alkali-soluble silica in the tufa, trass, or 'Moler' will enter into combination with the formed calcium hydrate, so that this, instead of crystallizing as such, crystallizes as mono-calcium silicate, after this formula:

$$Ca(OH)_2 + SiO_2 + H_2O = CaO, SiO_2, 2\,H_2O.$$

This is the same chemical process which gives the strength to the sand-lime brick, made of 98 parts of sand (silica) and two parts of slaked lime; only, in this case, the silica in the sand is not soluble, and a prolonged storage in superheated steam of 8 atmospheric pressure is required to effect the combination.

"The alkaline water of the desert countries contains, very often, potassium sulphate ($K_2\,SO_4$) and sodium sulphate ($Na_2\,SO_4$); these salts are undoubtedly destructive to Portland cement in the same manner as is the magnesium sulphate in the sea water, and it is therefore reasonable to assume that the admixture of trass, tufa, or 'Moler' to the straight Portland cement will be as beneficial for concrete work in these countries as it is for sea-water works."

The paper entitled "The Effects of Alkali on Cement," by George G. Anderson, M. Am. Soc. C. E.,* mentions this very condition, and

* *Transactions*, Am. Soc. C. E., Vol. LXVII, p. 572.

in the discussion, attention is directed to remedies which may be found by using puzzolan cement rather than straight cement. There is, therefore, good reason not to condemn offhand this adulteration, which for certain purposes may prove superior to straight Portland cement and for others equally well adapted. The speaker is fully aware of the danger and the difficulty opened up by the introduction of two or more standards for cement. These difficulties, however, will gradually be overcome if the combination can prove its right to existence, that is, its technical and economical advantages. Mr. Mogensen.

RAPIER R. COGHLAN, ESQ.* (by letter).—The blended cement manufactured by the City of Los Angeles for use in the Aqueduct would have been spared many criticisms if a more fortunate name had been applied to it. The officials of the Aqueduct Commission, in their various papers concerning the product, have used indiscriminately the terms, tufa-cement, tufa-Portland cement, puzzolan-cement, and puzzolan-Portland cement. Some engineers, not aware of the real composition of the cement, have claimed for it many of the bad qualities of the artificial puzzolans, such as are manufactured from blast-furnace slag and hydrated lime; while others have claimed that it is a simple adulteration of Portland cement with clay or clay-like material, and therefore harmful. These claims are widely at variance with what is actually the case. Mr. Coghlan.

The term sand-cement, while ordinarily meaning a blend of pure silicious sand and Portland cement, is not of necessity restricted to such a combination, but can be applied to a blend of Portland cement with granitic, basaltic, or many other sands that contain other elements and minerals than quartz. Even in the case of the Aqueduct cements, the tufa had to be reduced to a coarse sand before it was possible to blend it with the Portland cement. For this reason, it would seem that the best name to adopt for such a blended cement would be sand-cement. The nomenclature allows of the use of many materials, which have been experimented with and found to be as well adapted to the process as tufa.

Experiments have been conducted by the U. S. Reclamation Service on the blending of Portland cement with basalt, basaltic tufa, rhyolitic tufa, granite, andesite, trass, infusorial earth, and sandstone. In every instance it has been found that excellent grades of sand-cements can be produced from these materials, cements, which, up to the present time, have shown little or no decrease in strength. Researches on limestones and quartzites have shown, that, excepting those instances in which the amount of material added to the Portland cement was less than 40% by volume, these materials are not suited to the process.

It is a geological fact that the ultimate products of the decomposition of igneous or volcanic rocks are clays. Experiments by eminent

* Mfg. Cement Chemist, U. S. Reclamation Service, Elephant Butte, N. Mex.

Mr. Coghlan

men in the Engineering Profession have shown the deleterious effect of the addition of clay to Portland cement. Some concrete failures have been traced to the presence of too much clay in the sand or gravel, so that engineers have become accustomed to setting a maximum limit on the quantity of clay allowable in sand which is to be used for concrete. The results obtained with the so-called tufa-cement do not indicate that clay or clay-like substances have been blended with Portland cement. Tests made by the U. S. Reclamation Service laboratory, on material from the Aqueduct plants, corroborate fully the data set forth by Mr. Lippincott; in fact, by using a brand of cement (which was a recognized standard brand) other than that used at the Aqueduct plants, the results were greatly exceeded.

Careful chemical analyses of the raw materials suitable for sand-cement manufacture show that they are all dependent for their fitness on the quantity of colloidal or active silica; and many of the materials tested by the Reclamation Service experimentalists show a higher content of colloidal silica than the Los Angeles tufa. Most of these materials are harder to pulverize than tufa, but the resultant cement has proven to be as good in every particular. So fruitful have been the results of the researches that three sand-cement plants are being operated or erected by the Service: at Arrowrock, Idaho, Lahontan, Nev., and Elephant Butte, N. Mex. When all three of these plants are in operation, they will have a combined daily output of 2 300 bbl. of sand-cement. Such a wholesale manufacture and use of sand-cement would not be permitted by the engineers in charge of the work if the product was considered unreliable or apt to deteriorate with time. At those places where the cement is being used it is proving its worth. Little or no difference is apparent between sand-cement concrete and Portland cement concrete. It must be mixed a little dryer, and it sets and hardens more slowly than Portland cement concrete. The forms come away clean from the set concrete, and the water that comes to the surface brings little or no cement with it. Daily tests are made on all concrete placed in the dams, and the results of the compressive tests show that it is amply strong for the great stresses put upon it.

Sand-cement is of economic importance to the engineer in the West, as it allows him to use a cheaper cement at a price a little more than half of that of straight Portland. Before undertaking its manufacture or use, two things must be settled: The Portland cement must be of undoubted quality, a standard brand passing all tests; and the blending material must contain colloidal silica and be in an almost unaltered condition.

Mr. Paul.

CHARLES H. PAUL, M. AM. SOC. C. E. (by letter).—This paper opens up the subject of the use of blended cements, which is of great

interest to all engineers engaged in work in the West, where high freight rates and long wagon hauls usually combine to make the cost of Portland cement excessive. Mr. Paul.

In the construction of the Arrowrock Dam—which is being built by the U. S. Reclamation Service to store the flood waters of the Boise River—about 550 000 cu. yd. of concrete will be laid, and the cost of cement is, of necessity, a most important item. The dam is about 22 miles above the City of Boise and 17 miles above Barberton, the nearest point on the Oregon Short Line Railroad. A railroad from Barberton to Arrowrock has been built by the United States Government, over which the freight rate on cement charged against the work is 23 cents per bbl. The commercial freight rate on cement from Utah mills to Barberton is $1.14 per bbl., from California points $2 per bbl., and from Kansas points, $2.09 per bbl.; so that the total freight charges on cement to the Arrowrock work are from $1.37 to $2.32 per bbl.

Laboratory tests on sand-cement have been carried on by the Reclamation Service for 6 years or more, and, during the winter of 1910-11, when the investigations in connection with the storage works on the Boise River were nearly finished, a series of these tests was started, using for blending material the native rocks in the vicinity of Arrowrock, particularly granite from the spillway site, a large quantity of which would have to be wasted during the construction period.

TABLE 11.—Average of Three Long-time Tensile Tests on Sand-cement Mortars (1 to 3, by weight, standard sand), Each Test being an Average of 5 Briquettes; and Comparison with Mortars made from Portland Cement used in the Manufacture of the Sand-cements and Tested under the Same Conditions.

Kind of cement.	7 days.	28 days.	3 months.	6 months.	1 year.	2 years.	3 years.	5 years.
Sand-cement.....	219	333	397	423	457	423	449	444
Portland cement.....	266	400	425	482	460	496	428	404

Blending material for sand-cement was river sand, bank sand, or granite, blended in each case with equal parts by weight of Portland cement.

TABLE 12.—Average Long-time Tensile Tests on 13 Brands of Portland Cements Used by the U. S. Reclamation Service During the Last Five Years.

Mortar briquettes, 1 to 3 by weight, standard sand.	7 days.	28 days.	3 months.	6 months.	1 year.	2 years.	3 years.	5 years.
	260	373	448	447	432	411	379	398

Mr. Paul. It is noticeable that there is not the marked decrease in strength in the case of the sand-cement briquettes as is shown here, and is so common with Portland cements.

TABLE 13.—SAND-CEMENT TESTS ON MATERIAL FROM ARROWROCK DAM SITE, ARROWROCK, IDAHO.

Blending material, spillway granite. Mix, 1 to 3 by weight with standard sand. Strength, in pounds per square inch.

TENSILE TESTS ON STANDARD BRIQUETTES.

Percentage of blending material.	7 days.	28 days.	3 months.	6 months.	1 year.
0	343	428	461	467	408
30	374	417	467	499	494
40	368	467	504	....	....
50	330	419	460	452	410
60	294	369	400	....	....
70	156	237	266	320	335

COMPRESSION TESTS ON 2-INCH CUBES.

0	1 077	1 969	3 720	3 744	....
30	1 631	2 454	3 171	4 392	4 656
50	1 471	1 763	2 729	3 121	3 712
70	458	938	1 215	1 256	1 560

In Table 13 the first line in each case (0% blending material) represents the Portland cement from which the sand-cement is made, tested under the same conditions.

Investigations of the actual use of sand-cement in structures which had been in service for 10 or 15 years, further tests in the local laboratory, and estimates of cost of installation and operation of the necessary equipment, finally led to the decision to use sand-cement, in so far as practicable, in the construction of the Arrowrock Dam, and to use for blending material granite from the spillway excavation.

A sand-cement plant, with a capacity of 1 000 bbl. per 24 hours, consisting of a crusher and sand rolls, rotary dryer, ball mill, mixing machine, and three tube mills, all electrically operated, with the necessary bins, hoppers, and conveying machinery, has been erected and has been in operation for about 2 months. The cost of this mill, complete, was about $46 000, itemized as follows:

Excavation	$1 500
Foundations	3 750
Erection of building, chutes, etc.	8 150
Equipment, including freight	23 000
Installation of equipment	7 850
Electrical work	1 750
	$46 000

The total output of the mill to date has been about 25 000 bbl., and about 20 000 cu. yd. of sand-cement concrete have been placed in the dam, up to the present time. Mr. Paul.

All incoming Portland cement is tested and accepted before shipment. All sand-cement is tested for fineness, blend, setting time, and tensile strength in 1:3 sand briquettes, and is subjected to the boiling test before being used in the work. Tests for fineness and blend are made every hour that the mill runs, or oftener if either needs correction, and the other tests are made on an average sample from each day's run. The blend now being used is 45% granite by weight, and the requirement for fineness is at least 90% passing a 200-mesh sieve. Standard Portland cement requirements are satisfied by the other tests, and no sand-cement is used until it has passed the 7-day tensile tests of 200 lb. per sq. in. The writer's experience with sand-cement confirms that of the author with tufa cements, that no sample tested has ever failed to pass the boiling test.

TABLE 14.—Mill-run Tensile Tests on Sand-cement Manufactured at Arrowrock.

All tests on 1:3 standard sand briquettes. Strength, in pounds per square inch.

Average of all (52) 7-day tests	258	Average of all (29) 28-day tests	332
Average of 10 lowest 7-day breaks	221	Average of 10 lowest 28-day breaks	304
Average of 10 highest 7-day breaks	297	Average of 10 highest 28-day breaks	366

None of the sand-cement tested has failed to pass either the 7-day or the 28-day requirements. Occasionally, briquettes have been broken at the age of 4, 5, or 6 days, and in every case yet tested these briquettes have shown a strength exceeding 200 lb. per sq. in.

TABLE 15.—Cost of Manufacturing Sand-cement.

Items	Unit cost of sand-cement, per barrel.
Granite delivered to crushers	$0.02
*Portland cement, including freight and storing	1.35
Handling and storing Portland cement	0.08
Labor, operating	0.10
Power and lights, including maintenance, etc	0.16
Installation, depreciation, supplies, repairs, etc	0.14
Total cost	$1.85

*Portland cement @ $2.36 per bbl., f.o.b., Arrowrock.

A representative cost of manufacturing 1 bbl. of 45% by weight blend of sand-cement at Arrowrock is given in Table 15, which includes depreciation, on the plant and installation, at a rate which

Mr. Paul. will wipe out the total cost at the time that a total output of 500 000 bbl. is reached. It does not include sacking, as most of the sand-cement will be used in bulk.

The use of sand-cement under actual working conditions brings out several characteristics that are different from those of Portland cement.

Comparing sand-cement concrete with Portland concrete, batch by batch, an experienced concrete man will note no difference, either in its appearance or in its behavior while being spread, but, after a run of an hour or two, in mass work, it will be noticed that an ordinary plastic or quaking mixture gives off water more readily than does Portland cement concrete, and that this water is clearer and apparently carries less cement than is the case with ordinary concrete.

A mushy or really wet mixture which is worked up continually shows, at the surface, an accumulation of sloppy material which is more marked than in Portland concrete and sets and hardens very slowly. In fact, it is the writer's opinion that, for this reason, extremely wet mixtures should be avoided, especially with sand-cement concrete.

Sand-cement concrete sets more slowly than ordinary Portland concrete, and its slowness in hardening is very marked. It is natural to assume that internal shrinkage stresses, due to setting and hardening, will be less on this account, but, at the present time, we have no other evidence on this point.

The writer has not noticed that sand-cement concrete adheres to the forms any more than Portland concrete, and as he has had no occasion to use sand-cement plaster, he has not had an opportunity to compare it with ordinary cement plaster.

The author states that tufa cement makes a denser and more impervious concrete than Portland concrete. The writer has heard this statement made before, both in connection with tufa cement and sand-cement, but has never found record of any experiments or definite evidence to that effect. It would be interesting if the author would discuss that point more fully, and give the members the benefit of any experiments along that line with which he is familiar.

It is natural for the ordinary construction man to look on a new product with prejudice, especially when it shows characteristics to which he is not accustomed; but, supposing, for the sake of making a comparison from another viewpoint, that sand-cement had been in general use for years and that Portland cement were the new product being introduced; would we look with favor on a cement which almost invariably showed a decrease in strength after a few months? Or one which not infrequently failed to pass the boiling test? Would we consider it a favorable sign that the excess water coming to the surface of concrete apparently brought with it an unusual quantity

of cement? Quick-setting cements are usually considered undesirable for use in mass work, and were we accustomed to the slow-setting and slow-hardening sand-cement, might we not look with suspicion on a quick-setting cement, especially if one of the quicker-setting Portlands happened to be the first of the new product to fall into our hands? Mr. Paul.

The use of sand-cement in mass work, where the requirements are enough to justify the installation of the necessary grinding machinery, where suitable blending material is available, and where the transportation charges on Portland cement amount to a considerable portion of its cost laid down, will result in a marked saving in construction costs, and will give a product which is at least the equal of the Portland cement from which it was made, in fact, one which, for ordinary requirements, is not open to the least suspicion.

O. W. Peterson, Assoc. M. Am. Soc. C. E. (by letter).—The use of Haiwee tufa cement in the lining of about 40 miles of ditch of 930 sec-ft. capacity, during the past 3 years on the Olancha Division of the Los Angeles Aqueduct, has given the writer considerable first-hand experience with this product. This cement has been used, not only for lining the ditch, but in building various structures, such as arched flume crossings of 33 ft. span, etc. In fact, tufa cement has been used in exactly the same manner as any other cement, except that account has been taken of its slow setting. Mr. Peterson.

This open ditch section is excavated with long-boom, power shovels to as neat a line as possible, but, necessarily, in trimming the ditch to a true surface behind the shovels, back-filling is generally required. In such a dry country as this, which has an annual rainfall of only 5 in., the use of a well-compacted back-fill was considered, and has proven sufficiently good construction.

Against this trimmed earth section the 6-in. concrete lining is placed, without the use of forms. The finished ditch is 11 ft. wide at the bottom, with a 1-ft. invert, and the side slopes are 1 to 1. The depth of the ditch is about 13 ft. The concrete side lining is placed in alternate sections, 12 ft. wide, only 2 by 6-in. board strips being used to support the edges of the first set of alternate panels put in. Necessarily, the placing of concrete on such a steep slope without forms, cannot, even with the best efforts, result in the greatest compactness. Thus the tufa cement concrete is subjected to a further test, but again experience has shown that the strength of the slabs is sufficient to carry the load even against a support partly or entirely of back-fill.

The usual mixes of concrete have been 1 to 5 and 1 to 6, depending on the aggregates used. Generally, an extremely clean gravel, ranging from 40 to 75% of stone on the $\frac{1}{4}$-in. screen, has been easily obtained, but at places it has been necessary to add crushed rock.

Mr. Peterson.

The ditch is plastered with an extremely thin coat of 1 to 3 plaster, and for this purpose tufa cement seems to be especially well adapted, due largely to its fineness.

The temperatures in this locality range from 110° in summer to 8° in winter. A slow-hardening cement must be carefully protected at either of these extreme conditions. In summer, fresh work is sprinkled constantly for 2 weeks and covered with boards or canvas. In winter, boilers are used to furnish hot water for mixing purposes, thus turning out a warm concrete. This precaution, together with a covering of 1-in. boards or several layers of canvas, is sufficient protection against frost. The plaster is also mixed with hot water in winter, but generally no plastering is done in mid-winter.

From this brief description of operations, one can realize how difficult is the test to which the concrete for this work is subjected; and yet the practical completion of this division and the actual test of a full ditch of water for the greater part of 20 miles of work for 2 years' time has proven that the canal is water-tight. As the side concrete is really made up of separate 12-ft. slabs, slight joint lines show when the ditch is empty, but the expansion of the concrete due to moisture closes these, as well as other cracks, as the ditch is filled. Horizontal cracks are very infrequent, and arise from numerous causes, chief of which is probably occasional carelessness in back-filling. As a proof of the imperviousness of the lining, the leakage of the 20-mile section referred to has been less than a miner's inch per mile. This new ditch has never had running or muddy water in it.

The concrete has no tendency to swell under the influence of moisture, except to such an extent as all concrete expands in passing from a dry to a wet condition. This is a most desirable property for ditch lining, as it insures water-tightness.

In order to determine the value of cement rich in tufa, a carload of 75% tufa cement was obtained from Haiwee, and 300 ft. of concrete lining was put in with the usual mix. The laboratory tests showed that this special tufa was very slow setting. Frequent comparative tests for hardness were made of the special 75% tufa concrete and the ordinary 50% tufa concrete, using a pick for the purpose. At first the special tufa was much the softer, but as weeks went by the two products approached the same degree of hardness. At the last visit, when the age of both concretes was 10 weeks, there was but little difference in hardness. In fact, two visiting engineers were unable to detect any difference. This test seems to be proof of the value of Haiwee tufa. There are no perceptible cracks or indications of disintegration in the 75% tufa cement concrete. It will be watched with a great deal of interest.

In the lining of open canals, where proper care can be taken of the fresh concrete, slowness of hardening is no inconvenience. A saving

of from 40 to 60% in the cost is sufficient consideration to warrant the extensive use of tufa cement. No doubt the use of long-boom, power shovels to excavate canals, and the lining of the section with tufa cement concrete could have wide application in the economical building of many large irrigation channels, resulting in a great saving of water now lost by seepage, and consequently a large extension of irrigated areas. Mr. Peterson.

About twenty-five cars of straight Monolith cement have been used at times when the Haiwee mill could not supply the demand. Concrete made with this cement sets faster, but, as far as has been observed, it shows no advantage in ultimate hardness and strength over that made with Haiwee tufa.

A comparative test was made with blocks of tufa cement concrete in the strongly alkaline waters of Owens Lake and in fresh water. On removal, the concrete block which had been in the lake water 6 months was sound and apparently unaffected by the alkalis. It was not different in appearance from the block which had been in fresh water. However, this is not considered conclusive, as only from careful and extended tests can a thoroughly reliable conclusion be reached.

Luigi Luiggi, M. Am. Soc. C. E. (by letter).—From time immemorial, pozzolana, which is a variety of tufa, has been used in Italy for preparing mortar capable of setting under water. Mr. Luiggi.

The usual proportion is one volume of slaked lime and two volumes of pozzolana, mixed very intimately. Generally, the mixing is done by hand, but, lately, it has been done by machinery, which grinds the pozzolana more finely and mixes it more intimately with the lime. In this latter case the mortar is far superior, and sets more quickly, than when the mixing is done by hand. This is because the machinery makes a much more intimate mixture of the ingredients, and the silica of the pozzolana can act more directly and quickly on the hydrated lime.

All the harbor works by the Romans, which have lasted until now, for instance, that at Porto Trejano, at Porto d'Anzio near Rome, and Porto Adriano near Bari, were built with mortar, either for masonry or for concrete, made as indicated; and even now all, or nearly all, sea works in Italy are made in the same way, and always with excellent results.

However, pozzolana has one drawback: it sets very slowly, from 10 to 15 days in summer—when the heat accelerates the chemical reaction of pozzolana on lime—to from 15 to 30 days in winter. As it never freezes in Italian harbors, construction work is carried on throughout the winter, but, during that season, the setting of the mortar is very slow.

For accelerating this setting, Portland cement, in the proportion of 100 to 200 kg. per cu. m. of pozzolana concrete, is often used. The

Mr. Luiggi.

setting then takes place in 24 hours, or even less, and thus the concrete stands better the washing of waves, when it is laid under water or is used for bag-work.

This concrete has always stood perfectly the chemical action of salt water, as the silica of the pozzolana neutralizes all the free lime, which is inevitable even in the best Portland cement. There is even an excess of silica, which makes the mortar rather acid—not basic—and thus the action of the sulphate of magnesia of the sea-water is better counteracted.

The necessity of having a mortar of greater resistance, and more rapid setting qualities, however, has induced Italian harbor engineers to adopt Portland cement mortar and concrete. The writer was perhaps the first to use it in 1882 in some parts of the Harbor of Genoa. In those days, as Portland cement was rather a novelty for Italian engineers, the mortar was made rather rich in cement, at least in the proportion of 1 cement to 2 of fairly graduated sand with from 30 to 35% of voids, and often even 1 to 1. These mortars, and the concretes made with them, have stood the action of salt water very well. On the other hand, with mortars made afterward, with a smaller proportion of cement, or with sand rather fine, that is, with more than 35% of voids, the results have been bad, and, in some cases, disastrous. This the writer attributes to the porosity of the mortar, which, under the shock of waves, or the action of tides, permits the salt water to penetrate and percolate through the mortar and decompose it. On the other hand, when the mortar is rich in cement, and as impermeable as it is possible to make it, the action of salt water is nil, or at least so much diminished as to cause no bad consequences.

Therefore, the writer always recommends his engineers to use at least 10% more Portland cement than the quantity necessary to fill all the voids of the sand, and the extra expenditure for cement is only a very small fraction of the cost of the mortar or of the concrete, and absolutely negligible in comparison with the damage that might result by the decomposition of the mortar.

There is always, however, the latent danger of the free lime, unavoidable in Portland cement, being acted on by the sulphate of magnesia, with the consequent decomposition of the mortar. It was for this reason that the writer, in 1888, used, and has recommended ever since,* the addition of some pozzolana in Portland cement mortars and concretes, in order to neutralize this free lime. In this he took advantage of experience on the Roman sea-works. If they lasted some eighteen centuries, certainly the addition of pozzolana to Portland cement will correct the dangers of the free lime.

*"La Pozzolana ed il Cemento Armato nelle Opere Marittime," *Giornale del Genio civile*, 1910.

Mr. Luiggi.

Experience in Italy since 1888 has been uniformly good, and now pozzolana is being used for improving the concrete for the construction of a very large graving dock in Venice, which will be 850 ft. long, 120 ft. wide at the entrance, with 40 ft. of water on the sill.

The composition of the concrete for this very important work is shown in Table 16.

TABLE 16.

Components for 1 cu. m. of concrete.	Concrete in contact with sea-water for a thickness of about 6 ft.	Concrete for the inner mass of the graving dock.
Portland cement, in kilogrammes....	400	250
Pozzolana, in liters.....	150	70
Sand, in liters.....	450	440
Broken stone, in liters.....	1 000	1 000

This really gives a little more than 1 cu. m. of concrete.

The practice of adding pozzolana, or its equivalent in trass, or tufa, to Portland cement for sea-works is becoming rather common. It is followed in Holland and Germany, and was adopted also by the Japanese in the construction of the Nagasaki Dry Dock, and, thus far, the results have been uniformly good.

Pozzolana, or trass, or tufa, are volcanic materials of very little cost, the transportation being the only real drawback to their more general use.

Besides neutralizing the free lime of Portland cement by the large quantity of silica in these materials, they tend also to make the concrete less permeable, by the colloidal action of the alumina they contain.

The usual proportion recommended by the writer is about $\frac{1}{2}$ to $\frac{1}{3}$ of pozzolana for 1 volume of cement, the higher proportion being for Roman pozzolana, containing from 51 to 53% of silica, and $\frac{1}{3}$ for Naples pozzolana, which contains from 62 to 65% of silica.

This addition of pozzolana, besides adding very little to the cost of Portland cement concrete, makes it less permeable and eliminates the danger of decomposition in sea-water. The experience on Roman works made with pozzolana concrete and exposed to salt water for more than eighteen centuries is a guaranty of the soundness of this practice.

Thus the use of tufa cement, or Portland cement reground with tufa, as practiced on the Los Angeles Aqueduct, is quite correct, both with regard to the cost and the durability of the work.

The mortar and the concrete will certainly be less permeable than if ordinary Portland cement had been used, and impermeability is an all-important condition in all aqueducts, especially in one of such magnitude as that of Los Angeles.

Mr. Lippincott.

J. B. LIPPINCOTT, M. AM. SOC. C. E. (by letter).—The writer is gratified at the generous number of contributions that have been presented in the discussion of this paper, especially as arguments have been given vigorously on both sides of the question. Before entering into a review of the discussion, it is desirable to add certain data which have become available since the paper was written.

On page 530 there is given a test (up to an age of 6 months) of a mortar made of 75% of hydrated lime and 25% of tufa. The 1-year break shows a strength of 252 lb., an increase of 27 lb. over the 6-months break. This briquette shows a continued hardening in water, and demonstrates the hydraulic properties of the tufa in combination with lime.

Table 4 gives the strengths of Haiwee and Fairmont tufa cements up to an age of 1 year. Unfortunately, briquettes for longer time tests were not kept during the early stages of the work on the Los Angeles Aqueduct. A series is now being held for this purpose, but the 2-year period has not yet been completed. For briquettes made of tufa cement consisting of 50% of tufa and 50% of Monolith cement, mixed with 3 parts of sand, four breaks, made at the Haiwee mill, indicate a growth in strength from 500 lb. at the end of 1 year to 595 lb. at the end of 1 year and 289 days. Breaks of the oldest briquettes have just been made at the Fairmont plant, with the following results, each figure representing an average of ten tests. The cement used was 50% of Monolith and 50% of Fairmont tufa mixed with 3 parts of standard sand:

3 days.	7 days.	28 days.	3 months.	6 months.	1 year.	1 year, 319 days.
153 lb.	240 lb.	388 lb.	467 lb.	492 lb.	555 lb.	565 lb.

More breaks were not made because it is desirable to preserve the series for testing at regular periods. They indicate the continued hardening of the tufa mortar with age, which is characteristic.

Table 5 shows tufa sand briquette tests with varying proportions of tufa for ages up to 6 months. Table 17 gives results in continuation of that table.

The cement used (in the tests in Tables 5 and 17) was Monolith, and the tufa was from the Monolith quarries. A good increase in strength is shown between the 6-months and the 1-year breaks. Especial attention is called to the strengths shown in the 1-year breaks of the 80 and 85% tufa cements.

On page 544 is recorded the results of tests with a blend of Monolith cement and diatomaceous earth in the proportions of 50% of each. It should have been stated that this blended cement was mixed with 3 parts of sand.

TABLE 17.—TUFA SAND BRIQUETTE TESTS, WITH VARYING PROPORTIONS OF TUFA: A CONTINUATION OF TABLE 5.

Briquette number.	Percentage of tufa.	Tensile Strength.					
		3 days.	7 days.	28 days.	3 months.	6 months.	1 year.
21	55	85	200	335	370	330	410
		90	210	345	370	395	400
22	60	30	100	300	460	390	520
		35	100	310	435	385	460
23	70	35	90	250	350	330	460
		35	100	260	355	365	430
24	75	15	75	210	300	325	370
		25	80	200	285	375	270
25	80	15	75	150	255	300	415
		20	80	160	235	305	395
26	80	15	70	90	140	230	310
		20	70	90	110	260	265

Table 10 states the results of tests made with a mixture of volcanic clinkers and lavas found on the Hawaiian Islands with Santa Cruz (California) cement in equal parts by volume. Few tests were available for longer periods than 28 days at the time of writing the paper. Table 18 contains the results of tests for periods up to 1 year.

TABLE 18.—SAND BRIQUETTE TESTS MADE WITH A MIXTURE OF HAWAIIAN LAVA AND SANTA CRUZ CEMENT: A CONTINUATION OF TABLE 10.

Number of tests.	Date made.	Percentage water.	Tensile Strength.						Brand.	Percentage of lava.	Fineness.		Setting Time.				Boiling test, 6 hours.
			3 days.	7 days.	28 days.	3 months.	6 months.	1 year.			100 M.	200 M.	Initial.		Final.		
													H.	M.	H.	M.	
2	1911. Nov.	10½	140	255	340	450	465	480	.....	0	93.8	....	2	15	6	30	O. K.
8	Dec.	10½	112	190	307	400	467	470	Red Clinker.	50	....	92	2	30	6	00	"
0	1912. Jan.	10½	168	148	230	295	388	391	Lava.	50	....	92	1	..	11	..	"

The tests in Table 18 show fair results, and indicate a cement suitable for ordinary hydraulic work. This cement is not as strong as the blends made with the tufas already described. The tests show a uniform growth in strength with age.

Some tufa cement was sent to Robert E. Ford, Professor of Mechanical Engineering of Throop Polytechnic Institute, at Pasadena, Cal.,

Mr. Lippincott.

with a request that it be tested for compression. Standard briquettes were made and first broken in tension and the two halves then crushed. Cubes, 2 in. square, were also crushed, and the ratio of tension to compression was obtained. An unusual quantity of water was used by Professor Ford to get normal consistency (28½% for neat cement), and this, possibly, may account for the fact that the values are lower than those usually obtained in the city laboratories. The cement was Monolith, and the tufa was from Fairmont. Standard sand was used. Table 19 is a summary of the tests. The results for compression compare favorably with tests given by Taylor* for Portland cements, but these cements are not as strong as the German trass cements (the tests of which are given in Table 9), nor are they as strong as those mentioned in Table 13, of materials tested at the Arrowrock Dam, as quoted by Mr. Paul, either for their blended material or for their straight cement mortars.

TABLE 19.—Compression Tests of Los Angeles Aqueduct Tufa Cement (Monolith).

Briquette No.	Cube No.	Days cured.	Ratio: Tension-Compression.	Average Tension.	Average Compression.	Average ratio: Tension-Compression.
Neat Briquettes.						
5a.....	....	7	6.8	270	.....	....
.....	5m	7	7.6	...	1 945	7.2
4.....	....	28	8.0	...	.....	....
5b.....	....	28	8.0	...	.....	....
6a.....	....	28	7.3	...	.....	....
6b.....	....	28	9.3	490	.....	....
.....	5n	28	8.95	...	.....	....
.....	6m	28	8.7	...	4 125	8.4
5c.....	....	180	11.4	...	.....	....
6c.....	....	180	14.0	540	.....	....
.....	4m	180	15.35	...	.....	....
.....	6p	180	13.75	...	7 370	13.6
3 to 1 Standard Sand.						
7a.....	....	28	4.55	...	.....	....
7b.....	....	28	4.3	...	.....	....
8a.....	....	28	5.2	...	.....	....
8b.....	....	28	5.5	265	.....	....
.....	7m	28	4.0	...	.....	....
.....	7n	28	4.1	...	.....	....
.....	8m	28	4.6	...	.....	....
.....	8n	28	4.65	...	1 220	4.61
7c.....	....	180	6.25	...	.....	....
8c.....	....	180	5.95	...	.....	....
8d.....	....	180	5.5	367	.....	....
.....	7p	180	5.8	...	.....	....
.....	8p	180	6.9	...	1 225	6.1

* "Practical Cement Testing," p. 215.

Mr. O'Hara states that Professor Eakle, of the University of California, considers that "the rock ground with the cement used on the Los Angeles Aqueduct, is a rhyolite-tuff and a trachyte-tuff, which, when finely ground, will possess the same characteristics as finely-ground clay." Mr. Lippincott.

Mr. O'Hara continues:

"The cement produced by the method used at Monolith and Haiwee, Cal., is similar to cement adulterated with clay. Rhyolite-tuff and trachyte-tuff are not to be confounded with the volcanic rocks known as puzzuolana and trass, which have been used for the manufacture of cement. Puzzuolana and trass are the hardened products of volcanic action in their original state, in which respect they are similar to blast-furnace slag. On the other hand, the volcanic tuff of the nature found at Monolith and Haiwee, is not comparable to blast-furnace slag, being more of the nature of altered volcanic rock."

Table 2 gives an exact chemical analysis of the tufas used on the Los Angeles Aqueduct, of Italian tufa, and also of Italian puzzuolana (as used in that country), as determined from samples furnished by the American Consul. A detailed analysis of the German trass is given in Table 20 where it is shown that it has a close resemblance to the California tufas. The local clays analyze much more closely to the Italian puzzuolanas than the California tufas. The percentage of silica in the local tufas, soluble in alkalies, is greater than that in the samples of Italian puzzuolanas, in the ratio of 4 to 1, according to determinations by the Monolith Laboratory, and this is believed to be a very important factor. Mr. O'Hara is mistaken when he refers to these tufas as an altered volcanic rock. If he had ever seen these quarries, he probably would not think so. The tufa used is an unaltered volcanic ash which has been thrown down in water and comminuted. It has been partly consolidated by pressure. Physically, it has no more resemblance to clay than unground cement clinker, which also contains chemically a high percentage of clay. It resembles somewhat the pumice stone which can be purchased at most drug stores. Many of the hardest rocks, such as granite, slate, and lava, on decomposition form clay, and it would be as misleading to say that a granite is a clay as to say that this tufa is a clay. The writer is familiar with tufa in its altered and clay-like condition. It is found near Haiwee.

The tufas are very low in sulphur, containing usually less than one-tenth of 1%, and it is difficult to see how there is any extra danger because of being blended with straight cements containing 1½% of sulphur, on this account, as Mr. O'Hara states. Reference is made to the discussion by Mr. Luiggi as to the enduring quality of these cements in sea water.

Mr. Lippincott.

Mr. O'Hara also says:

"Tufa and Portland cement, as used on the Los Angeles Aqueduct, is a mechanical mixture, the two materials being blended in equal parts by volume. Under these conditions, no silicates of lime are found, and the gradual increase in strength is not due to the same cause as the slow hardening of a high silica Portland cement."

The chemical analyses of the Bureau of Standards, given on page 537, as well as the analyses at the laboratories of the Los Angeles Aqueduct, show that silicates of lime are formed between the tufa and cement. Mr. O'Hara's statement, as given, is only empirical, and should not outweigh these chemical determinations. The writer regrets that Mr. O'Hara considers the practice of blending tufa cements on the Los Angeles Aqueduct as "dangerous and without precedent." Many unprecedented things are done, especially by the Engineering Profession, which result in general benefit. There is comfort in the fact that the U. S. Reclamation Service is now building or operating three cement blending and re-grinding plants, after careful investigations, made by its engineers, as shown by the instructive discussions contributed by Messrs. Rapier R. Coghlan and Charles H. Paul.

Mr. Reed calls attention to the severe climatic conditions under which the concrete work on the Los Angeles Aqueduct is carried on. The relative humidity in the Mojave Desert during the summer is about 20%, and this, together with temperatures which are often greater than 100°, and the great scarcity of water, makes a most trying condition for concrete work. Except on the 40 miles of open canal described by Mr. Peterson, methods of work were devised which kept the green concrete shielded from the sun and wind. Standing water is usually maintained in the conduit with small dams, to give an artificial humidity. Doors or curtains were maintained at tunnel portals. The roof was cast on the outside conduit before the forms were taken from the sides, and it was covered with wet earth on top as soon as the concrete was poured. The bottom was placed last, and the manholes were kept closed. It was only by such methods that shrinkage cracks could be avoided, no matter what kind of cement was used. In this way, mile after mile of the outside conduit could be built without any expansion joints and without transverse cracks. Leaving a manhole open for a week or two would often cause a transverse crack to open within 10 ft. Where the conduit has been filled since, these cracks, in many instances have swelled shut.

Mr. Richardson states that:

"Unfortunately, no data are available showing what proportion of the silica in any of the materials occurring abroad, or in those in California, is soluble in acid, as it is hydrated silica of this description which possesses the greatest hydraulic value."

Mr. Lippincott.

It is the writer's understanding that it is solubility in an alkali solution which is desired. This laboratory determination is made with a dilute solution of sodium carbonate. On page 537 the U. S. Bureau of Standards is quoted as showing that 7.17 out of 30.97 parts of the silica in the briquettes made of Fairmont tufa cement, or 23%, were soluble in the mortar. A test of the pure pulverized Monolith tufa, in the Monolith Laboratory, showed 8.4% soluble.

Comparative analyses of the German cements and the Monolith tufa are given in Table 20. The information in reference to the German materials (referred to in Table 9) is from the report of the Royal Testing Laboratory.*

TABLE 20.—Analyses of German Trass and Monolith Tufa.

	German trass.		Monolith tufa.
Loss on ignition	3.29	10.25	6.28
Water of crystallization	6.96		
Insoluble silica	41.56		
" "	1.39	68.49	69.48
Soluble silica	25.54		
Iron oxide	2.27		2.52
Aluminum	9.83		11.37
Lime	1.05		1.80
Magnesia	0.67		2.95
Potassium	3.04		Not determined.
Sodium	3.09		" "
Phosphorus	0.11		" "
Sulphuric acid	0.06		0.43
(Unable to translate)	0.10		....
	98.96		

These analyses show a striking similarity between the German trass and the Monolith tufa. The quantity of sulphur shown in Monolith tufa is unusually large in this sample, and the 69.48% is for total silica, no division having been made between soluble and insoluble.

The sample of Italian tufa available at the Monolith Laboratory shows only 2% of silica soluble in dilute sodium carbonate. This sample was furnished through the American Consulate, and possibly may not be typical. What was furnished as Italian puzzuolana shows practically the same.

Replying further to Mr. Richardson, the tufa is separately ground in the ball mill, so as to pass through a 20-mesh screen, but the final grinding of the blended tufa and cement is done in the tube mill. An effort was made to grind the cement clinker with the crushed tufa, but it ground very slowly in the tube mill. As it was also desirable to save transportation charges, the final grinding and blending were done mostly at distant points on the work, where tufa was available, thus saving half of the freight.

Mr. Wagoner quotes from a printed "Report on Municipally Manufactured Cements of the Los Angeles Aqueduct" made to the American

* Mitteilungen aus dem Königlichen Materialprüfungsamt, 27 Vol., 1909, p. 348.

Mr. Lippincott.

Portland Cement Manufacturers Association, by a party of five engineers. The circulation of the report has been suspended by the Association. As an example of its bias, an illustration is given of a test slab which was not a portion of the Aqueduct, and had been loaded to destruction by the city's engineers, in order to test the concrete materials, as used in construction. In the pamphlet this is labeled "Slabs of poor concrete, where cover on L. A. Aqueduct has already collapsed and been replaced, on desert west of Mojave." The writers of that report state that there are three kinds of concrete on the Aqueduct: good concrete made of purchased cement; fair concrete made of Monolith cement; and poor concrete made of tufa cements. They state the places where these concretes are found, and it is amusing to note that their "poor concretes" in most cases actually were made of purchased cements.

The Monolith mill has been run for four years. The product, with minor exceptions, has been standard. The hydraulic index which Mr. Wagoner has taken exception to, has usually been about 2.0. It is not claimed that the mill is infallible. Neither are other mills. It has been watched closely by cement manufacturers, and when it was in temporary trouble, due to a change in the limestone quarry, it was promptly investigated. The writer regrets that it is necessary to refer at all to this report. Much more could be said, but it would involve local politics and business foreign to this discussion. Table 21 contains an independent analysis of Monolith cement.

Mr. Wagoner suggests that tests with tufa and some standard cement be made for the Profession. Many barrels of tufa cement have been made with both Colton and Riverside cement, and they show the same results as those made with Monolith cement.

In reply to Mr. Mogensen, it may be stated that no discriminations have been made against tufa cements. Two reinforced concrete towers have been successfully built of tufa cement, as well as several flumes, and nine reinforced concrete pipes, 10 ft. in diameter, for heads up to 80 ft. Many miles of reinforced roof slab, 8 in. thick, with a span of 11 ft. 5½ in., have been successfully laid. The Haiwee Dam referred to is an earthen structure with a clay core; no concrete was used in its construction. Numerous prominent engineers have examined the work, and, although most of it is in a remote and unoccupied region, every effort has been made to aid investigations by the Profession.

Approximately, the following quantities of cement of various classes have been used to date on the Los Angeles Aqueduct:

Monolith standard cement	900 000 bbl.
Tufa cements	636 000 "
Purchased standard cements	260 000 "

Mr. Lippincott

TABLE 21.—Comparisons of Standard Cement Specifications with Tests of Monolith (Los Angeles City) Cement, Furnished by the Park Department, as Tested by Smith Emery Company, February, 1913.

Physical Tests.

Authority.	Tensile Strength, in Pounds per Square Inch.				
	Neat, Pure.			1 part cement and 3 parts sand.	
	1 day.	7 days.	28 days.	7 days.	28 days.
American Society for Testing Materials. Specifications	175	500	600	200	275
U. S. Bureau of Standards. Specifications	None	500	600	200	275
Smith Emery Company Test	182	632	798	202	273
	216	709	800	214	285
	244	656	838	215	280

Time for 28-day test not yet expired.

Chemical Tests.

	U. S. Bureau of Standards. Specifications. Percentage.	American Society for Testing Materials.	Tests by Smith Emery Company.		
			No. 1.	No. 2.	No. 3.
Silica	19 to 25	.....	25.02	24.72	24.80
Alumina	5 to 9	.....	5.58	5.40	5.52
Iron oxide	2 to 4	.....	2.72	2.00	2.48
Lime	60 to 64	.....	62.70	63.20	62.92
Magnesia	1 to 4	Not more than 4	1.76	1.98	1.66
Sulphur trioxide	1 to 1.75	Not more than 1.75	1.14	0.95	1.08
Loss on ignition	0.5 to 3.0	.....	1.80	2.00	2.00
Insoluble residue	0.1 to 1.0	.....	.....	.....	.....
Fineness, through 200-mesh sieve	75% or more	75% or more	81.2	80.4	80.8
Fineness, through 100-mesh sieve	92% or more	92% or more	94.0	94.0	94.2
Setting time	Not less than 45 min	Not less than 30 min.	3 hours.	3 hours.	3 hours.
Boiling test	Boil without cracking.	Boil without cracking.	O.K.	O.K.	O.K.
Specific gravity	Not less than 3.10.	Not less than 3.10.	3.12	3.12	3.12

The ratio of the weight of the aluminum silicate to that of the lime is called the Hydraulic Index, and should be as high as 1.91, the German cements showing 1.98. This analysis indicates for Monolith cement 2.08. (Note by J. B. Lippincott.)

Mr. Lippincott.

The contributions to the discussion by Messrs. Coghlan and Paul are of particular interest and value. Little may be added to these discussions; they are commended to those interested in this subject. The writer agrees with Mr. Coghlan's statement, referring to materials suitable for blending, "that they are all dependent for their fitness on the quantity of colloidal or active silica"; and that they should "be in an almost unaltered condition." Mr. Paul's tests, showing the results of blending the granites at the Arrowrock Dam of the U. S. Reclamation Service, are especially interesting, as they broaden the opportunity for this economy. The writer's experience with the lavas from the Hawaiian Islands, though very limited, is in accord with that of Mr. Coghlan.

It must be constantly borne in mind by Eastern engineers, in considering this subject, that the prevailing price of cement at central points in the Far West, is about $2 per bbl., and that transportation charges are high. Engineers in this region are anxious to build the necessary works with the available funds. The saving on the Los Angeles Aqueduct, due to the use of tufa cement, will amount to about $700 000.

Mr. Peterson's discussion gives the experience of an engineer who has used tufa cement for three years, and, for this reason, it is of value.

Mr. Luiggi, who is Inspector General of the Royal Civil Engineers (of Italy) and Professor of Maritime Construction at the Royal School of Civil Engineers, gives a most interesting discussion of the subject, referring both to the old Roman cements, made of lime and puzzuolana (tufa), and the blending of Portland cement with puzzuolana. He has practiced this latter blending since 1888. He approves the practice, and states that it is "quite correct, both with regard to the cost and the durability of the work."

Mr. O'Hara quotes Professor Eakle, of the University of California, as stating:

"The rock ground with the cement used on the Los Angeles Aqueduct, is a rhyolite-tuff and a trachyte-tuff, which, when finely ground, will possess the same characteristics as finely ground clay."

Mr. O'Hara then makes the deduction that "the cement produced by the method used at Monolith and Haiwee, Cal., is similar to cement adulterated with clay." Presumably, he gets this information from the pamphlet issued by the Association of American Portland Cement Manufacturers. The writer, having communicated with Professor Eakle and called his attention to the paper and to Mr. O'Hara's discussion, received, under date of February 27th, 1913, the following:

"Your letter of February 26th, informing me that I had been quoted by cement manufacturers was news to me, and so I have looked up the

quotation to which you refer. I am rather in the dark regarding the matter, as both the gentlemen using my name and the rock in question are unknown to me. However, I constantly receive samples of rock for identification, and always try to accommodate the sender with whatever information I can give, and as to the persons sending me material seldom give me the localities or state what they wish to do with my report, I only hear of it later by some roundabout way as this." Mr. Lippincot

After receiving this communication, the writer forwarded samples of the tufa from the Fairmont quarries to Professor Eakle for indentification and for an expression of opinion. He was requested to put this in such form as to permit of publication. The following letter has been received in compliance with this request:

"The samples of tuff you sent me cannot be classed as strictly fresh rocks, as the original more or less glassy base has become devitrified, a portion of the alkalies leached out, and the tuff is considerably hydrated. The alteration has, however, been more in the nature of an incipient kaolinization and silification producing a hardened stony base, as shown by the gritty powder, so the value of the rock for the use you make of it has not materially suffered by the slight change. The amount of alumina in the tuffs is rather high and there is no doubt that an extra amount of clay-like material is added to the cement, but your experiments and those of German cement users seem to prove conclusively that such rocks do not act as adulterants, but, on the other hand, produce a better and stronger cement, if properly used. Much of the free silica and alumina silicate of these tuffs is in a more soluble state than it is in ordinary clay and quartz sand, and it is presumably due to this fact that the tuffs have their value. Their solubility is shown by treatment with HCl and with alkali carbonates, and is also shown in the field where many of them have become silicified and often impregnated with opal, deposited from the colloidal silica formed. When these tuffs are ground so intimately with cement, every portion of this soluble silica and silicate comes into contact with the lime, and during the gradual drying out of the moist mass, a chemical action is taking place to form lime silicate. The longer you can keep the interior mass moist, the more complete should be the crystallization and the stronger the mass. In quick setting we would expect the tuffs to act more as an adulterant. It would be interesting and of value to know if thin sections of the cement at different periods of solidification would not show what is taking place. The tuffs you sent are rhyolite tuffs. Your article states you use equal parts of cement and tuffs, but the analysis of the Monolith tuff cement shows a proportion of about one part tuff to three parts cement. There is a discrepancy in these analyses. The tuff cement has more water than the tuff and the amount of alumina is not larger than some Portland cements contain, and the total allows nothing for alkalies which must be present.

"I would like to offer a suggestion about the name 'Tufa Cement' which you use. It has become the custom with petrologists and

Mr Appincott

geologists, at least in this country, to use the word 'tufa' for calcareous deposits, and 'tuff' for volcanic ash rocks; so it might be better to call the cement a 'tuff-cement.'

"You are perfectly welcome to make use of anything in this letter you may find of assistance."

With reference to Professor Eakle's comment on a discrepancy in the analyses, it should be remembered that the pulverized tufa is blended with the cement at the mills in equal parts by volume. The ratios are: 60% by weight of cement to 40% by weight of tufa, approximately. The mixture for this sample, the analysis of which is given in Table 2, was of equal parts by volume. The analyses, of course, show percentages by weight. According to the writer's computations, there is the following relation between the analyses given and the theoretical quantities which should occur:

	Silica.	Iron and alumina.	Lime.
Theoretical quantities....	41.1	10.87	38.19
Analysis	35.34	11.89	41.05

The percentages shown in these different analyses, either for the cement or for the tufas, will vary from time to time, and cannot be expected to check absolutely. The quantity of silica in the tufa used in this blend is unusually high.

Mr. O'Hara also quotes from the U. S. Corps of Engineers,* as follows:

"Puzzolan cement never becomes extremely hard like Portland, but puzzolan mortars and concretes are tougher or less brittle than Portland. * * * It is unfit for use when subjected to mechanical wear, attrition, or blows."

In order to test the tufa cements for attrition, twenty pieces of briquettes of tufa cement (50% by volume being tufa and 50% by volume Monolith cement, mixed with 3 parts of standard sand) which had attained an age of one year, were revolved in the small laboratory ball mill with flint pebbles; and twenty pieces of broken straight cement briquettes which had been tested for tension, which were a year old, and had been mixed with 3 parts of standard sand, were treated in a similar manner. The tensile strength of the briquettes of tufa cement was about the same as those of straight cement, each set having averaged more than 400 lb. The briquettes were weighed separately before being subjected to the rattler test, and were weighed at equal intervals of time as the test proceeded. The briquettes of tufa cement were run separately from those of straight cement, because it was difficult to distinguish between them after the grinding had proceeded for some time. The results are given in Table 22.

* Professional Paper No. 28, p. 1779.

TABLE 22.—ATTRITION TESTS OF TUFA CEMENT AND STRAIGHT CEMENT BRIQUETTES MADE WITH 3 PARTS OF STANDARD SAND. Mr. Lippincott.

Time	5 minutes.	20 minutes.	50 minutes.	2 hours.	5 hours.
Straight	5.26%	10.85%	14.23%	28.007%	48.13%
Tufa	3.13%	7.66%	13.93%	28.9%	63.41%

The percentages represent the loss in weight.

This test indicates that, for periods up to 50 min., the tufa briquettes stood abrasion better than those of straight cement; that, for the 2-hour run, they were practically the same; and that on the 5-hour run the straight cement showed a superiority of about 30 per cent. It has been established in the tensile strength tests, that the tufa is slower in hardening than the straight cement, and it would appear that the outer portion of the tufa briquettes had attained a greater hardness than those of straight cement, but that the interior was not yet as hard.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1255

THE SEWICKLEY CANTILEVER BRIDGE OVER THE OHIO RIVER.*

By A. W. Buel, M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Le Conte, Charles Worthington, Theodore A. Straub, C. W. Hudson, V. R. Covell, Henry H. Quimby, and A. W. Buel.

Introduction.

The titles of half a score of papers presented to this Society bear the familiar names of great cantilever bridges. Of such bridges in the United States, five exceed the Sewickley Bridge in weight of steel per linear foot, and four exceed it in length of channel span; but some details and erection methods used in this bridge are thought to have sufficient merit and novelty, at least in their application, to justify the hope that a description largely confined to such features may prove interesting.

The Sewickley Bridge has a channel span of 750 ft. from center to center of towers. This is exceeded in the United States by the Blackwells Island Bridge, 1 182 ft.; the Wabash Bridge over the Monongahela at Pittsburgh, 812 ft.; the Memphis Bridge, 790 ft., and the Beaver Bridge, 769 ft. The Mingo Bridge and the Thebes Bridge both have shorter channel spans, 700 ft. and 671 ft., respectively.

The weight of steel, including the buckle-plates under the roadway floor, is 3.48 tons per linear foot of bridge. This is about 70% of the weight per foot of the Thebes Bridge; 77% of that of the Wabash Bridge at Pittsburgh, and more than 99% of that of the Memphis

* Presented at the meeting of November 20th, 1912.

DISTANT VIEW.

Bridge. When the weight of the concrete foundation and block pavement of the 28-ft. roadway and of the reinforced concrete sidewalk slabs, 5 ft. 7¾ in. wide on each side, is added to that of the steel, it will be seen that the ratio of dead load to total load is probably greater than in any other cantilever bridge in the United States, with the exception of the Blackwells Island and Beaver Bridges.

The ratio of dead to total loads made it necessary to take great care in providing for the erection stresses, which determined the sections of an unusually large number of both truss members and details. The connections of the floor system and the sub-panel members, as required for the finished bridge, were entirely inadequate to take the traveler reactions or the 30-ton locomotive crane, which weighed 75 tons. The main panel point pins were practically the only details of sufficient strength to take the traveler reactions, and the expense and difficulty of increasing other members and details to carry them would have been very great.

The illustrations will be largely relied on to show the general dimensions, stresses, sections, and ordinary details, but an attempt will be made to present a few points of interest in shop plans, by which economies in weights and shop costs were effected, and particularly some details and provisions for erection which gave very satisfactory results.

Historical.

In 1894 a movement was started for the construction of a bridge over the Ohio River between the Boroughs of Sewickley and Coraopolis. Notwithstanding the fact that there was then no highway bridge over the river above Wheeling, 100 miles from Pittsburgh, the Board of Viewers reported that the probable traffic did not then justify the expenditure.

The movement was renewed in November, 1906, by a petition from the inhabitants of the sections concerned, resulting in an investigation by a new Board of Viewers. This Board reported to the Court that there should be a bridge at this point, but that the cost would be greater than it was reasonable for Sewickley Borough and Moon Township to bear. The approval of this report by the Court and Grand Jury gave authority to the County Commissioners to construct the bridge, if, in their opinion, it seemed expedient. The Commissioners then ordered the construction of the bridge, on April 10th, 1908, and appropriated

funds for the work during the following year. The principal considerations which resulted in this action were:

First.—That there was no highway bridge over the Ohio River above Rochester, which is 25 miles below Pittsburgh.

Second.—That Sewickley is approximately midway between Rochester and "The Point" (at the junction of the Allegheny and Monongahela Rivers) where both streams are spanned by bridges.

Third.—That Sewickley and Coraopolis are the largest towns on the Ohio River in Allegheny County.

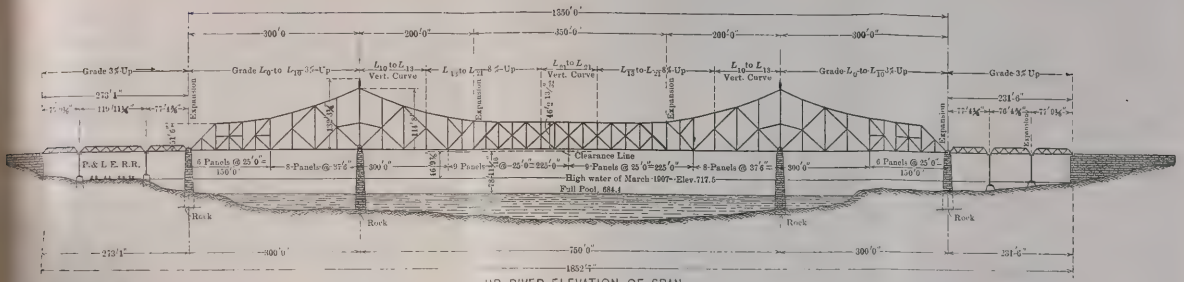
Fourth.—That the bridge would be a connecting link between road systems on each side of the river, comprising many miles of improved roads. At the present time there are 30 miles of improved country roads on the north side of the river and 40 miles of such roads on the south side, tributary to this bridge.

Following the authorization by the County Commissioners, surveys were made and contract plans and specifications prepared under the direction of Mr. J. G. Chalfant, County Engineer. The contract plans comprise 27 sheets, showing general details and arrangements of all parts of the substructure and superstructure. The sheets showing the substructure were, practically, final details. The sheets showing the general details of the superstructure were generally to a scale of $\frac{3}{4}$ in. to the foot.

Surveys.

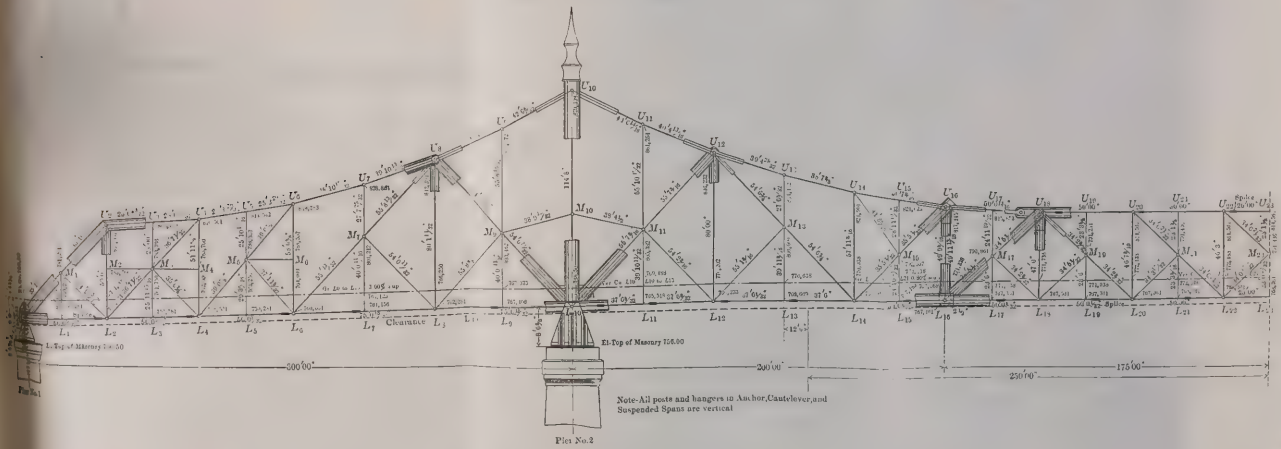
The preliminary surveys were made during the spring and summer of 1906, and were sufficient to fix accurately the length of the spans and locate the piers and abutments, together with the grades on the approaches and over the bridge. The center line was monumented on both sides of the river during June, 1909, and a triangulation system was laid out. That this part of the work was done with a high degree of skill and accuracy was proven by the satisfactory way in which all parts of the work came together in the field; and it reflects great credit on the County Engineer and his assistants. The triangles closed with errors of one part in 77 000 to one part in 192 000, and to within one or two seconds of arc. The Bridge Company checked the channel span by direct measurement before erecting the south half of the suspended span, and found it to be correct.

S 86A



UP RIVER ELEVATION OF SPAN

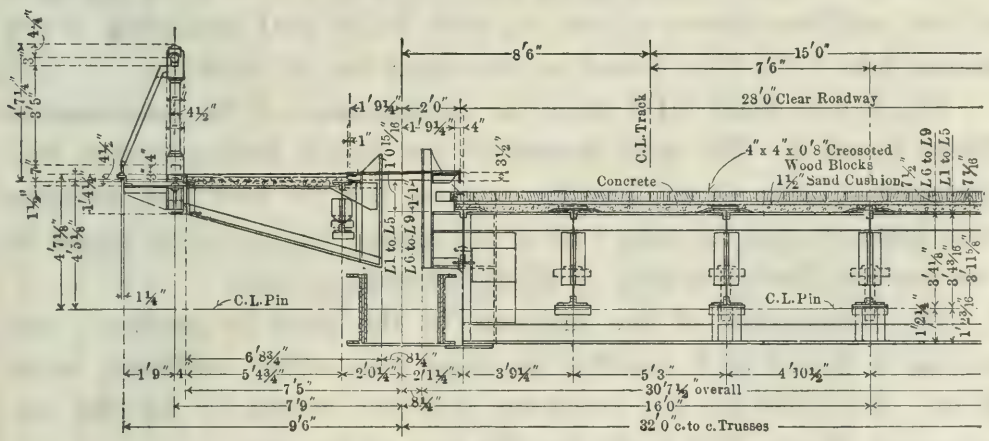
SECTION AT TOWER



Note-All posts and hangers in Anchor, Cantelever, and Suspended Spans are vertical

Contract Plans.

Outline.—Plate XII is a skeleton elevation of the structure, with dimensions and elevations, prepared in the County Engineer's office. These dimensions were strictly adhered to in the construction. The length of the channel span, the direction of the axis of the bridge in relation to the current at low stage, and the clearance above high water, were practically fixed by the regulations of the Federal Government. The outline and many of the general details of the trusses, shown on the contract plans, were similar to those of the Wabash Bridge, mentioned previously; but, in the latter, the main towers on the cantilever piers are made up with two columns in each truss, connected by horizontal bracing in the plane of the truss, without diagonals; whereas, in the Sewickley Bridge, these towers have only one column in each truss.



HALF CROSS-SECTION THROUGH FLOOR AT PANEL POINT L_6
THIS SECTION IS TYPICAL, EXCEPT FOR SIDEWALK FASCIAS AND STRINGERS

Fig. 2.

Floor System.—The floor system of the Sewickley Bridge was necessarily very different from that of the Wabash Bridge, as the latter is a double-track railroad structure, whereas the Sewickley Bridge is a highway structure, carrying a 28-ft. roadway with two electric car tracks and two 6-ft. sidewalks on brackets outside of the trusses. The trusses are 32 ft. from center to center. The roadway floor consists of a 4-in. wood block pavement on 1½ in. of sand, with concrete filling, averaging about 3 in. thick, over the ⅜-in. buckle-plates, which were placed with buckles down, and supported by stringers. The sidewalks are reinforced concrete slabs, with an average thickness of about 5 in., supported on two lines of stringers. Fig. 2 is a cross-section of half of the roadway and sidewalk.

Substructure.

The substructure comprises two abutments with wing-walls, four pairs of pedestals, two on each side, supporting the approach spans, the two anchor piers, and two cantilever piers. The foundations of the South Anchor Pier, No. 1, and of the two Cantilever Piers, Nos. 2 and 3, were carried to rock, which was found at elevations of from 30 to 35 ft. below "Full-Pool" water level. The work was done with coffer-dams, without unusual difficulties. The North Anchor Pier, No. 4, was founded on a very dense clay and gravel formation at Elevation 706.4 (about 20 ft. below "Full-Pool" level).

All piers were faced with sandstone and backed with 1:3:5 gravel concrete. The stone for Piers Nos. 1, 2, and 4 came from Wampum, Pa., and for the others from Morgantown, W. Va.

Granite bridge seats, 8 by 11 by 3 ft., were provided for the shoes of the cantilever towers to rest on, each bridge seat consisting of two stones, 4 by 11 by 3 ft., joined on the center line of the truss.

Fig. 3 and Plate XIII show the arrangement of the anchorage in Piers 1 and 4. The steel flanges of the double lattice girders were designed to take all the possible stresses, but the steel web members were proportioned for only half of the shear, reliance being placed on the concrete for about 50% of all the shearing stresses.

The appearance of the masonry in the piers is excellent, and, during a period of 9 months, the piers had settled uniformly about $\frac{3}{4}$ in. Mr. Chalfant, his assistants, and the contractor for the substructure, deserve full credit for this part of the work.

Loading.

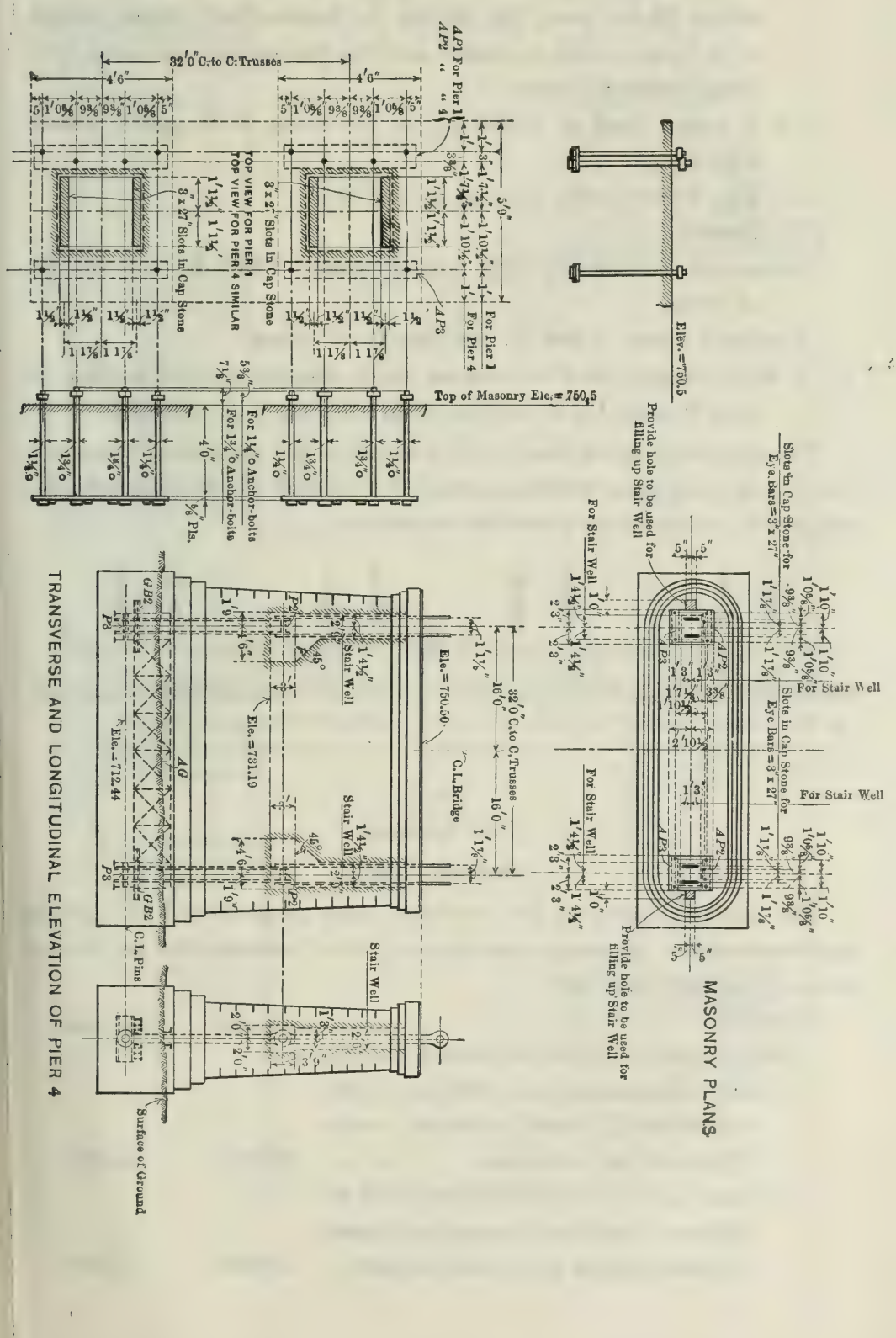
Dead Loads.—The dead loads consist of the weight of the structure itself, including that of the floor system, of any special features peculiar to the structure, and of the street-car rails.

For the contract plans, the dead load was assumed to be concentrated with two-thirds on the loaded chord and one-third on the unloaded chord. In the final computations, made by the Bridge Company, the actual concentrations were computed and used.

Live Loads.—

(A) Floor System.

(a) A uniform load of 100 lb. per sq. ft. of clear roadway and footwalk.


32'0" C. to C. Trusses
4'6"
4'6"
TOP VIEW FOR PIER 1
TOP VIEW FOR PIER 4 SIMILAR
8 x 27" Slots in Cap Stone
Top of Masonry Ele. = 750.5
For 1¼" ø Anchor-bolts
¾" Pls.
Elev. = 750.5
Provide hole to be used for filling up Stair Well
Slots in Cap Stone for Eye bars = 3" x 27"
For Stair Well
MASONRY PLANS
Ele. = 750.50
Ele. = 731.19
Ele. = 712.44
C. L. Bridge
C. L. Pins
Stair Well
Surface of Ground
TRANSVERSE AND LONGITUDINAL ELEVATION OF PIER 4

(b) A 15-ton road roller arranged as follows: 6 tons on the forward axle and 9 tons on the rear axle; axles 11 ft. apart, rollers 20 in. wide, the two on the forward axle being placed 2½ ft. from center to center, and the two on the rear axle, 6 ft. from center to center.

(c) A wagon load of 10 000 lb. on two axles, 8 ft. apart, wheels 5-ft. gauge.

(d) Two 50-ton street cars, as in Fig. 4.

(B) Trusses.

Suspended, cantilever, and anchor spans, 1 600 lb. per lin. ft. of truss.

Approach spans, 2 000 lb. per lin. ft. of truss.

A wind pressure of 35 lb. per sq. ft. of exposed vertical surface on both trusses, the structure being considered as unloaded.

The maximum stress due to wind added to the maximum stress due to live and dead loads was not permitted to exceed 20 000 lb. per sq. in., reduced by compression formulas for struts.

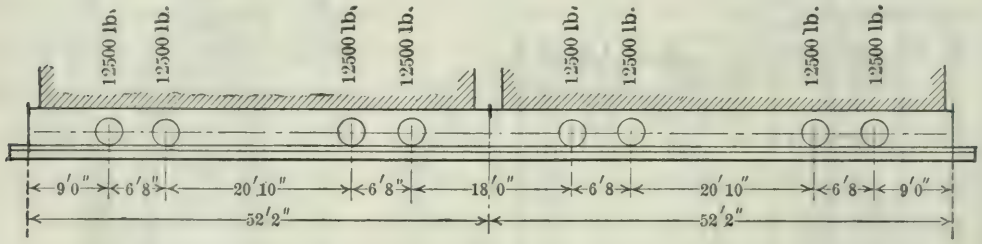


FIG. 4.

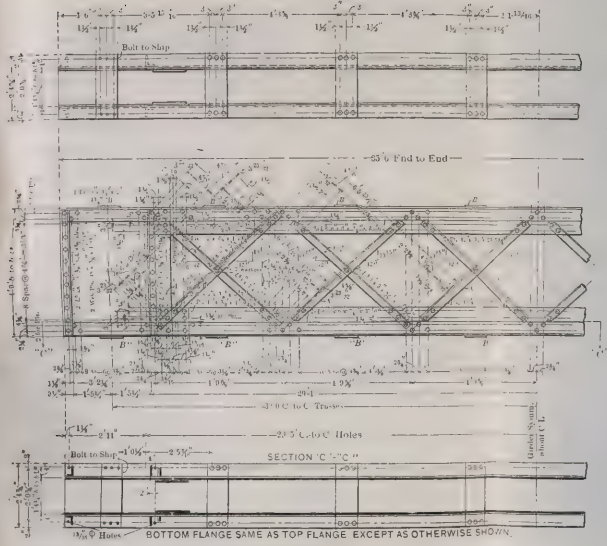
UNIT STRESSES.

(Medium Steel.)

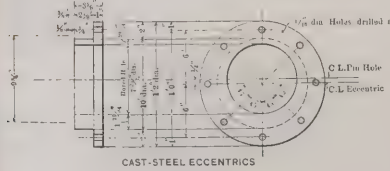
The different parts of the structure were proportioned so that the maximum stress, in pounds per square inch of section, did not, in any case, exceed the following:

Tension.—	Dead loads.	Live loads.
Net sections of shapes in bottom chords, main diagonals, and other parts not enumerated, forged eye-bars, main diagonals, and counters.....	24 000	12 000
Extreme fibers of rolled sections used as stringers and floor-beams, or flanges of plate girders when used as such..	12 000	12 000

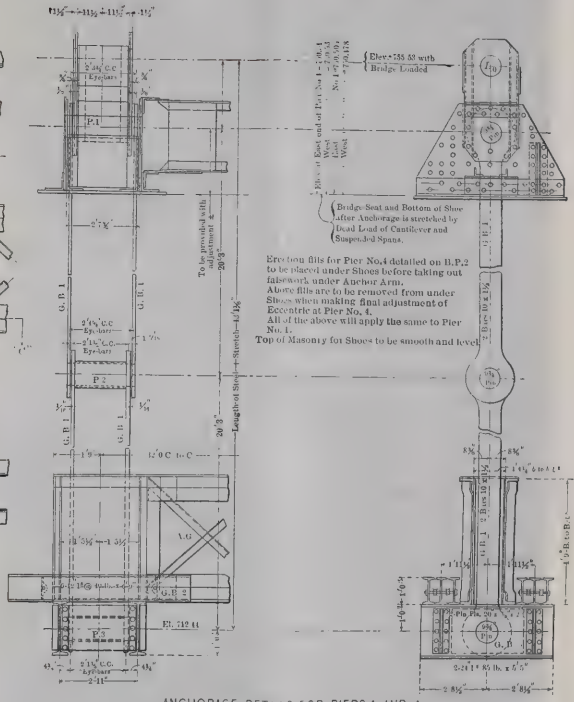
PLATE XIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1255.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



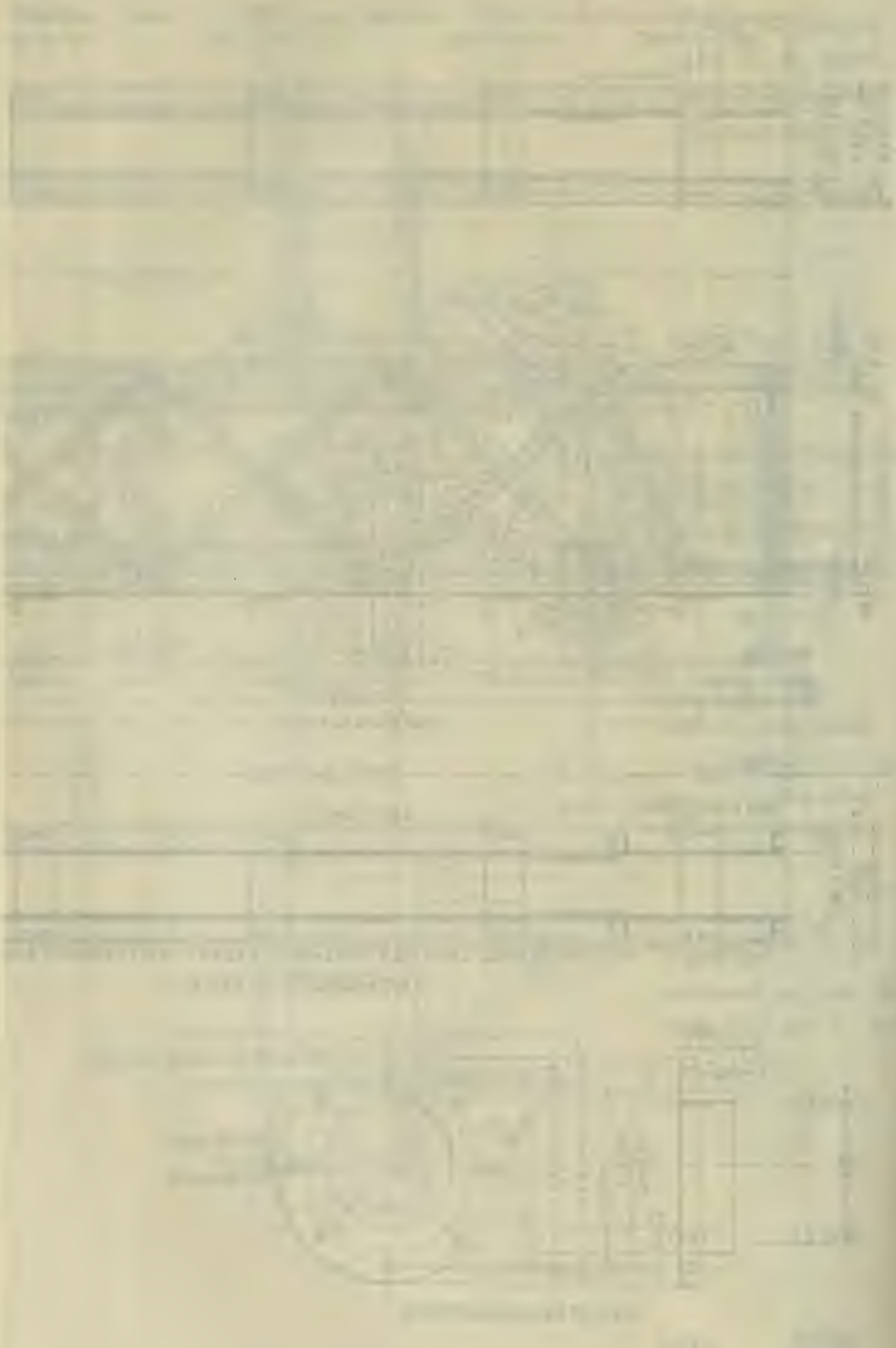
ANCHORAGE GIRDERS



CAST-STEEL ECCENTRICS


Elev. 755.53 with Bridge Loaded
Bridge Seat and Bottom of Shoe after Anchorage is stretched by Dead Load of Cantilever and Suspended Spans.
Erection fills for Pier No. 4 detailed on B.P. 2 to be placed under Shoes before taking out falsework under Anchor Arm.
Above fills are to be removed from under Shoes when making final adjustment of Eccentric at Pier No. 4.
All of the above will apply the same to Pier No. 1.
Top of Masonry for Shoes to be smooth and level.
To be provided with adjustment
Length of Steel
12'0" C to C
El. 712.44

ANCHORAGE DETAILS FOR PIERS 1 AND 4



	Dead load.	Live load.
Net sections of shapes or eye-bars in suspenders, hip verticals, hanger yokes, or other members subject to sudden loading	20 000	8 000

Compression.—

	Dead loads.	Live loads.
Chord members	$24\,000 - 80\frac{l}{r}$	$12\,000 - 40\frac{l}{r}$
Intermediate and end posts of through and deck trusses, and columns in trestle bents	$20\,000 - 90\frac{l}{r}$	$10\,000 - 45\frac{l}{r}$
Laterals and rigid bracing	$13\,000 - 60\frac{l}{r}$	

l = length of the member, in inches.

r = least radius of gyration of the section, in inches.

No compression members have a length greater than 120 times their least radius of gyration in the direction of possible flexure, except for the struts in the wind-bracing systems, which were limited to 150 times their least radius of gyration in the direction of possible flexure.

Bending.—	
On extreme fibers of pins closely packed	25 000
Bearing.—	
On pins	22 000
On shop rivets	20 000
On expansion rollers, per linear inch (where D = diameter of roller, in inches)	$500D$
On bridge-seat masonry—sandstone	500
On bridge-seat masonry—granite	1 000
Shearing.—	
On pins and shop rivets	10 000
On webs of plate girders used as stringers and floor-beams	5 000

The unit stresses on the shop rivets of the floor system and their connections did not exceed 80% of the foregoing for all kinds of stress. This applies also to the field rivets in the main girders and trusses. The unit stresses on all the field rivets of the floor system and bolts did not exceed two-thirds of the foregoing for all kinds of stress. The unit

stresses on the rivets in the lateral and sway-bracing were increased by 40% of the foregoing for all kinds of stress.

The other sections of the specifications conformed to the best practice for steel bridgework, requiring sub-punching and reaming, and all details of workmanship were of the highest grade.

The contracts were awarded about July 1st, 1909, to the Fort Pitt Bridge Works, of Pittsburgh, Pa., for the superstructure, and to Adam Laidlaw and Company, for the substructure. The contractor for the superstructure will hereinafter be referred to as the "Bridge Company." Ground was broken on July 21st, and actual work was started on the following day.

Preliminary Work of the Bridge Company.

The Bridge Company concluded to have the engineering work, including the most important of the detailed plans, executed by a specially selected squad of their own engineers and draftsmen, who would devote their entire attention to this particular job, under the supervision of the writer, who was specially employed for this work, and afterward retained as Consulting Engineer during erection. Before this squad could be gotten together, some progress was made by the regular engineering force of the Bridge Company, which took up the work immediately. They first proceeded with a careful revision of the estimated weights and dead loads, and then made graphical check analyses of the stresses. At this point the special squad took charge.

Checking and Revising Computations and Sections.

From the results of the graphical check analyses and other considerations it seemed that the weight of that part of the anchor piers directly over the anchorage might possibly not have a sufficient margin of safety, and, if the greater part of the mass of masonry above the elevation of the anchorage could be made absolutely effective, there would be a considerable excess. As some progress had already been made on one of the anchor piers, any change in the anchorage had to be decided on immediately, or serious delay would result. It was suggested, therefore, that heavy lattice girders, extending nearly the entire length of the pier, should be embedded in the concrete so as to bring practically the entire weight of the superimposed masonry into positive action to resist the uplift. This suggestion was adopted, with the

PLATE XIV.
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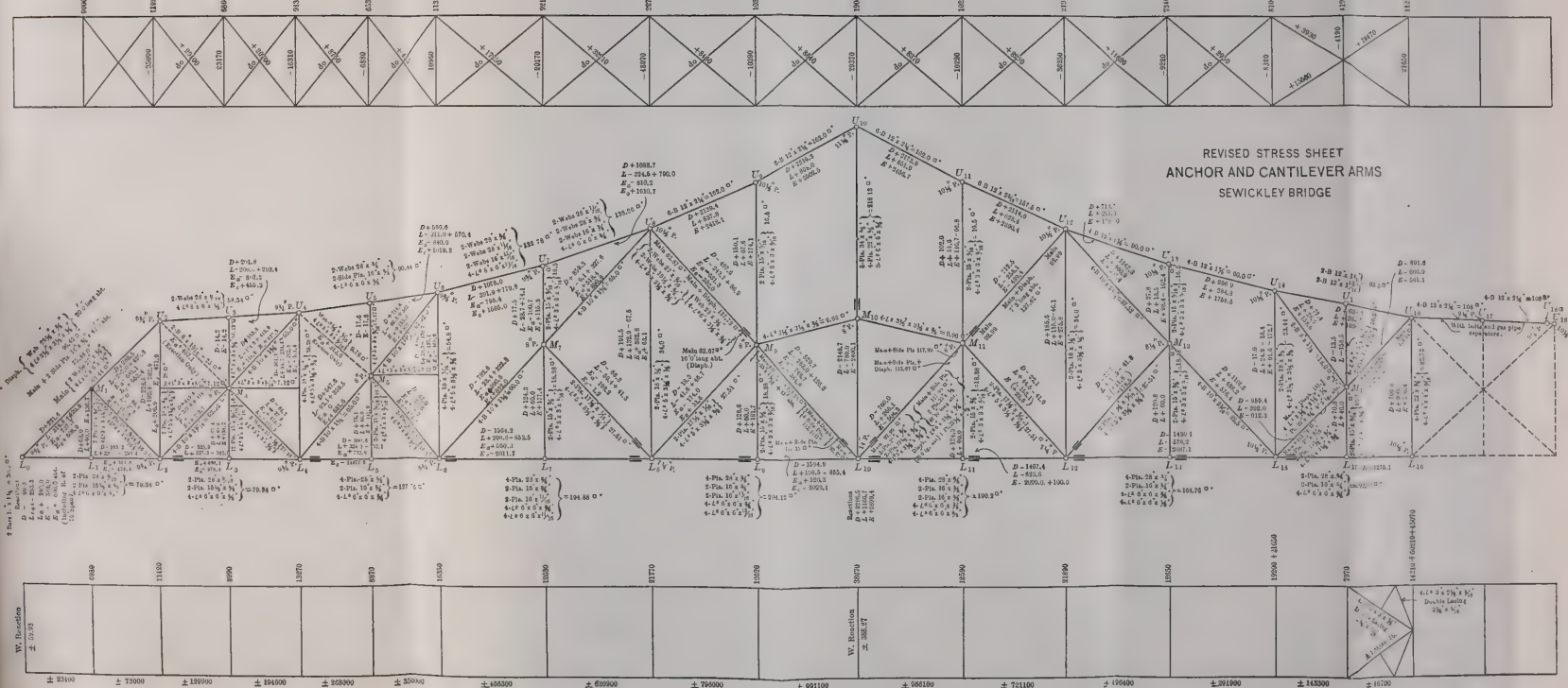
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.

REVISED STRESS SHEET
ANCHOR AND CANTILEVER ARMS
SEWICKLEY BRIDGE



incidental advantage of reinforcing the piers so that no danger of cracks need be apprehended.

The next step was to check and review thoroughly the dead-load determinations and compute the actual concentrations at each main and sub-panel point (at each point of intersection).

Then tables of stress coefficients, giving the stress in each member for a unit load at each and every point, were prepared and carefully checked. With the tables of stress coefficients and the revised dead-load concentrations, the stresses in each member for the loads at each point were computed and tabulated. These tables were arranged so that the maximum stresses were readily determined by taking summations between limits, which the tables indicated clearly by the plus and minus signs.

The stresses due to wind were then computed and tabulated, the wind load at each panel being based on an accurate estimate of the exposed areas. The wind shear from the top chord was assumed to go down to the lower chord in the planes, U_{18} to L_{16}, U_{16} to L_{14}, U_{12} and U_8 to L_{10}, and U_2 to L_0, only two panels of wind load being considered as going down the tower, U_{10}-L_{10}.

The final results of this work proved the accuracy of the stress computations on which the contract plans were based, but indicated the advisability of making a few changes, and these were authorized subsequently by the County Engineer.

Top Chord of the Anchor Arm.—The sections and make-up of the end posts and top chords of the anchor arm called for by the contract plans, from L_0 to U_8, consisted of eye-bars (proportioned for the principal tensile stresses), packed inside and outside of a very light riveted member which latter consisted of two 28-in. built channels, latticed top and bottom. These riveted members, while probably sufficient for the slight compression to which they might be subjected in the end post, L_0-U_2, under certain conditions of loading in the finished bridge, were entirely inadequate for the erection loads, should it become desirable or necessary to swing the anchor arm before the cantilever arm was erected. The Bridge Company considered it very important to provide for swinging the anchor arm at this stage of the work. It is also difficult to pack eye-bars in the same member with a riveted chord so that all will work in unison. On presentation of these considerations, the County Engineer approved the change submitted by the Bridge Company,

whereby the end posts and top chords of the anchor arms, from L_0 as far as U_8, were made entirely of riveted tension members designed to carry compression during erection and, in the finished bridge, where the stresses reverse toward the end of the anchor arm. Some details of these members will be described later.

Lower Chord Sections.—Some increase in the lower chord sections between L_6 and L_{14} being indicated as advisable, in the light of the revised computations of stresses due to dead, wind, and erection loads, and as an increase in depth of the 28-in. webs would have involved considerable difficulty and delay at that time, these members were changed from two built channels (with four 6 by 6-in. angles) to two built I-sections (with eight 6 by 6-in. angles). This reduced the unsupported length of the top and bottom lattice bars, so that flats could be used instead of channels or angles.

The foregoing changes, together with some in minor members and details, required an entire revision of the dead-load concentrations. This work was expedited and facilitated greatly by the tables of stress coefficients. It was possible for two men to make and check an entire set of revised dead-load stresses for the anchor and cantilever arms in two or three days. As this had to be done several times, these tables saved more than their cost, in the work of computing stresses alone, even if their great value in calculating deflections be neglected. They not only saved time, but made it possible to utilize junior and clerical assistants for a considerable part of the work, which was almost entirely reduced to the simple mechanical operations of slide-rule multiplications and summations by adding machines.

The final stresses and sections are shown on Plate XIV, for the anchor and cantilever arms, and by Fig. 5, for the suspended span. The latter are practically identical with those shown on the contract plans, no material changes having been made, except in the diagonals, U_{22}, M_{23}, and L_{22}. The dead and erection loading for each panel is shown in Table 1.

Shop Details.

The revised stresses and sections having been approved by the County Engineer, the preparation of shop details and mill order bills was commenced. Each member and each main and sub-panel point was laid out to a scale of 1½ in. to the foot, beginning with the chords of the anchor arm and following with all the other members throughout

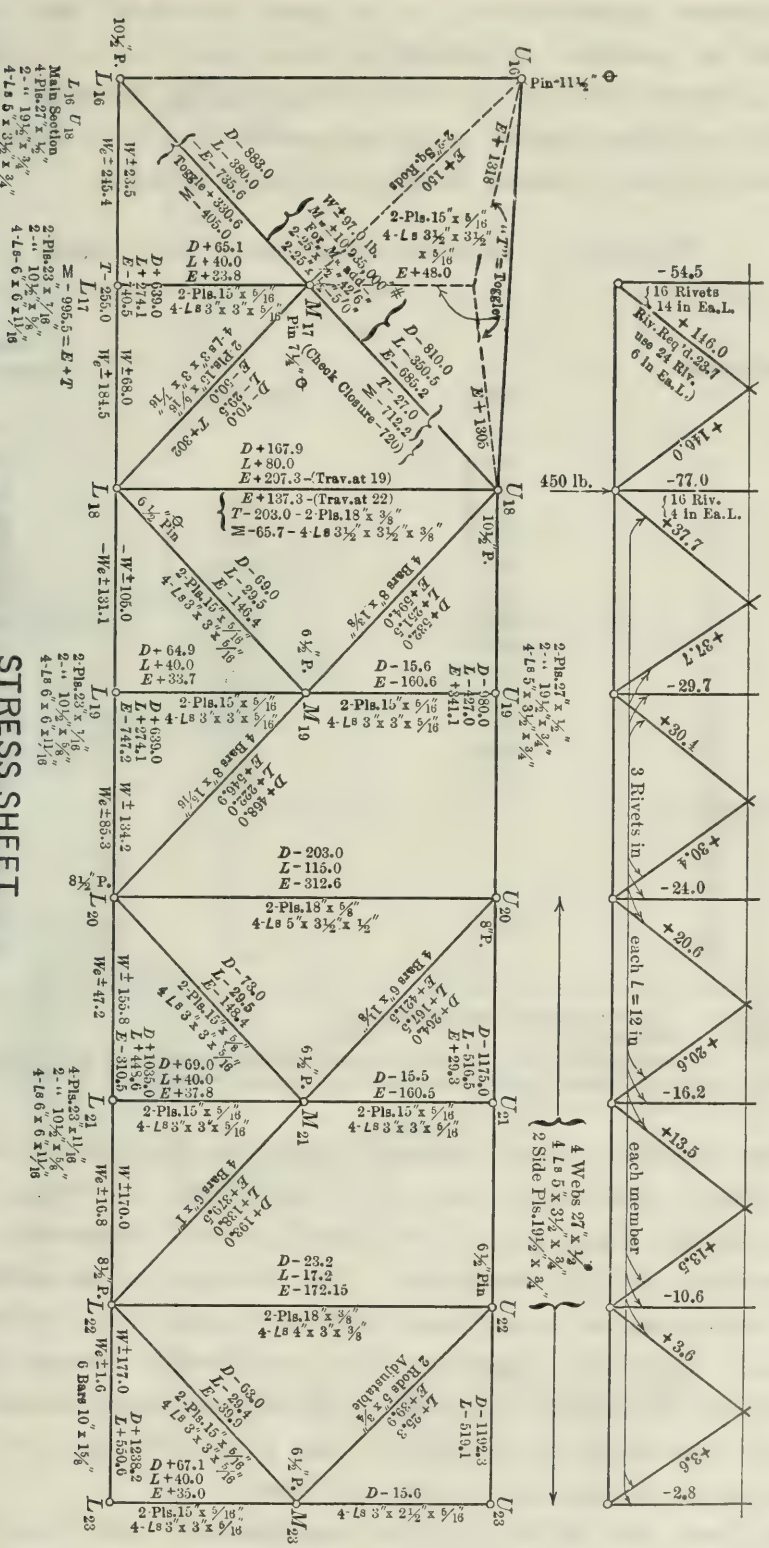

STRESS SHEET
ALL STRESSES IN THOUSANDS OF POUNDS
Pin 11¼"
Pin 7¼"
6½" Pin
8½" P.
8" P.
10½" P.
6½" P.
"T" = Toggle
Toggle +330.6
M -405.0
W ±97.0 lb.
M ±10,000,000
F 94.4
2-25" x ½" Pls.
4-Ls 5" x 3½" x ½"
E +48.0
E +150
E +1318
E +1305
D +65.1
L +40.0
E +33.8
D +167.9
L +80.0
E +207.3 -(Trav. at 19)
E +137.3 -(Trav. at 22)
T -203.0 - 2-Pls. 18" x ⅜"
M -65.7 - 4-Ls 3½" x 3½" x ⅜"
D +64.9
L +40.0
E +33.7
D +69.0
L +40.0
E +37.8
D +67.1
L +40.0
E +35.0
D -15.6
E -160.6
D -15.5
E -160.5
D -23.2
L -17.2
E -172.15
D -203.0
L -115.0
E -312.6
2-Pls. 18" x ⅝"
4-Ls 5" x 3½" x ½"
2-Pls. 18" x ⅜"
4-Ls 4" x 3" x ⅜"
450 lb.
4 Webs 27" x ⅜"
4 Ls 5" x 3½" x ¾"
2 Side Pls. 19½" x ¾"
each L = 12 in
each member
3 Rivets in
- 54.5
16 Rivets
14 in Ea. L.
+ 146.0
Riv. Req'd. 232.7
use 24 Riv.
6 in Ea. L.)
+ 140.0
-77.0
16 Riv.
4 in Ea. L.
+ 37.7
+ 32.7
- 29.7
+ 30.4
+ 30.4
- 24.0
+ 20.6
+ 20.6
- 16.2
+ 13.5
+ 13.5
- 10.6
+ 3.6
+ 2.8
- 2.8
Main Section
4-Pls. 27" x ½"
2-" 19½" x ⅜"
4-Ls 5" x 3½" x ¾"
2-Pls. 23" x 7/16"
2-" 19½" x ⅜"
4-Ls 6" x 6" x 11/16
2-Pls. 23" x 1"
4-Pls. 23" x 11/16"
6 Bars 10" x 1⅝"

Fig. 5.

the structure, approximately in the order in which they would be required in erection. Considerable pains were taken to treat each detail, however small, on its own merits, applying economic principles of design and shop practice as far as possible, but never losing sight of the risks and costs involved in the problems of erection.

TABLE 1.—Dead and Erection Panel Loads.

Panel points.	Dead.		Erection (steel only).
	Total.	Top (steel).	
0	55.78	.....	40.16
1	93.20	.....	61.95
2	107.14	33.49	75.91
3	94.15	14.24	62.91
4	112.27	35.98	81.02
5	103.30	17.60	72.07
6	151.37	45.15	112.32
7	173.12	30.83	126.25
8	193.33	51.99	146.46
9	197.99	30.62	151.12
10	391.55	193.90	344.68
11	193.28	29.83	146.41
12	184.05	46.75	137.18
13	161.22	17.43	114.35
14	138.03	27.73	98.98
15	108.33	13.51	77.09
16	117.92	22.34	86.68
17 to 23, inc.	611.19	.....	.....
17	.....	.....	72.91
18	.....	.....	67.65
19	.....	.....	56.81
20	.....	.....	65.17
21	.....	.....	58.37
22	.....	.....	60.08
23	.....	.....	54.27

Live load = 1 600 lb. per lin. ft. of truss.
Traveler = 72 500 at U_{11}.
" = 145 000 at U_{12} to U_{22}, at last panel point erected to give maximum stress.
Service track and loaded truck on anchor arm during erection = 100 000 lb. at any one panel point of truss.

Some difficulty was encountered in packing the members on the top chord pins of the anchor arm, due to the rather scant room provided by the contract plans, and also to some increase in thickness caused by the revisions already mentioned. Any increase in the width of the main truss members would have involved changes in practically every member in the bridge, together with a considerable increase in weight, and delay in completing the work. The built members forming the end post and top chord of the anchor arm, principally subject to tensile stresses, were worked out to pack as follows:

The lower end of the end post was designed with forks engaging the pin, L_0, between the webs of the lower chord and the rocker, the in-

PLATE XV.
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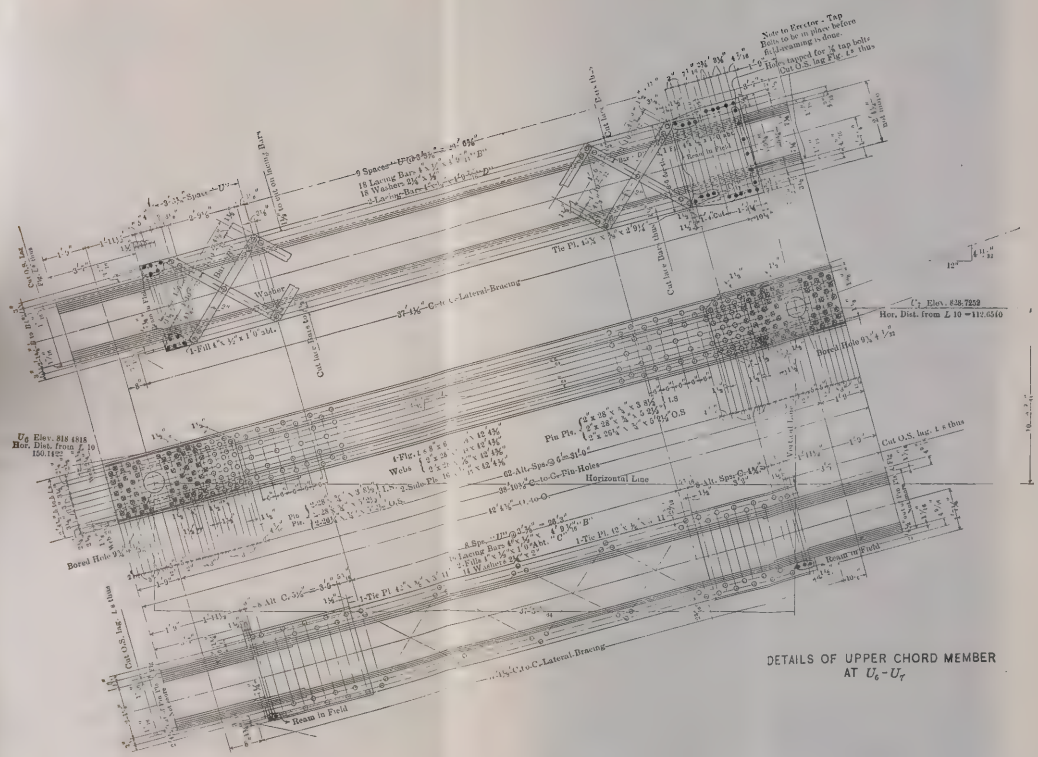
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE


Note to Erector.—Tap Rolls to be in place before full assembling is done.
Cut O.S. lug Fig. 1 a thus
Tie Pl. 1"
Cut bars flush
Bored Hole 9¾" ϕ 1/32
9 Spaces @ 11'0 3/16" = 99' 6 1/16"
18 Lacing Bars 4" x 1/2"
18 Washers 2 1/4" x 1/2"
2 Lacing Bars 4" x 1/2" x 1'-9" Bbt. "D"
3 Alt. Spaces @ 3'-3 3/16" = 9' 9 9/16"
Washer
Cut bars flush
Tie Pl. 4" x 2 9/16" x 1'-0"
37-14 1/2" C-to-C. Lateral Bracing
1-Fill 4" x 3/8" x 1'-0" Bbt.
Cut bars flush
Cut O.S. lug Fig. 1 a thus
Ug Elev. 818.4818
Hor. Dist. from L 10 = 150.1427
Ug Elev. 638.7259
Hor. Dist. from L 10 = 117.6510
Bored Hole 9¾" ϕ 1/32
4 Fig. 1 a
Webs
Pin Pls. 2" x 38" x 3 3/16" LS
2" x 38" x 3 3/16" LS
2" x 20¼" x 5 9/16" O.S.
2-Side Pls. 16" x 3 3/16" LS
62-Alt. Spaces @ 5'-3 1/16"
38'-10 1/2" C-to-C. Pin-Holes
Horizontal Line
Fig. 1-30 a
Fig. 1-30 b
Cut O.S. lug. 1 a thus
Vertical Line
6 Alt. Spaces @ 5'-3 1/16"
Cut O.S. lug. 1 a thus
12" x 12" x 1'-10 3/4" = 3'-3 3/4"
6 Spaces 11'-0 3/16" = 66'-1 1/8"
16 Lacing Bars 4" x 1/2" x 1'-9" Bbt. "C"
2 Fills 4" x 3/8" x 1'-0" Bbt.
11 Washers 2 1/4" x 1/2"
1-Tie Pl. 10" x 3/4" x 11'-0"
Bored Hole 9¾" ϕ 1/32
Beam in Field
1-Alt. C. 5 1/2" = 3'-0"
1-Tie Pl. 4" x 2 9/16" x 3/4" x 11"
Cut O.S. lug. 1 a thus
37-14 1/2" C-to-C. Lateral Bracing
Beam in Field

DETAILS OF UPPER CHORD MEMBER
AT U_6-U_7

to-in and out-to-out dimensions of the chord and rocker, respectively, being retained as shown on the contract plans. This fixed the dimension, in to in, of webs of the end post, and determined the dimension, in to in, of pin-plates for this member at U_2. This figure was adopted for the dimension, in to in, of pin-plates, for both ends of the top chord sections, U_4-U_6 and U_7-U_8. The dimensions, out to out, of pin-plates at both ends of U_2-U_4 and U_6-U_7, were then fixed at $\frac{1}{2}$ in. less than the in-to-in dimensions of pin-plates on the adjoining members, thus giving $\frac{1}{2}$ in. total clearance for packing. It will be seen that this arrangement resulted in the end post and two top chord members being wide and the other two intermediate top chord members being narrow; also, the dimension, back to back of webs, was variable, due to the variation in the thickness of the pin-plates required. In order to minimize the appearance of this difference in the width of the top chord members, those that packed outside were built up with 6 by 6-in. angles, whereas those that packed inside, and were consequently narrow, were built up with 6 by 8-in. angles, the 6-in. leg being always against the web.

The rivet spacing in the webs and pin-plates of these five members was worked out so as to keep the net section, through and back of the pin-holes, as large as possible and to give similar spacing in all pin-plates. The spacing through the body of the members is 6 in., alternating with a fractional space between the 6-in. spacing and the closer pin-plate spacing. Plate XV shows the upper chord, U_6-U_7, which is typical.

All the lattice bars, top and bottom, were made in two lengths, without any special bars. This result was obtained by varying the pitch slightly to compensate for the difference in dimensions between gauge lines, and by varying the length of the end tie-plates. This plan was followed throughout the work, an effort being made to keep the lattice bars of the same length for all similar members, and to avoid special bars. Tables showing the items of the lattice bars were then made, with sufficient data for the shop to prepare all the bars, ready for assembling. This was an appreciable advantage, not only in preparing the bars, but in assembling, as the shop plans designated the lattice bars for each member by their item numbers as given in the tables.

Camber and Finished Lengths of Truss Members.—It was desired to camber the bridge so that, under full dead and live loads, it would come to the exact position and elevations shown by the contract dia-

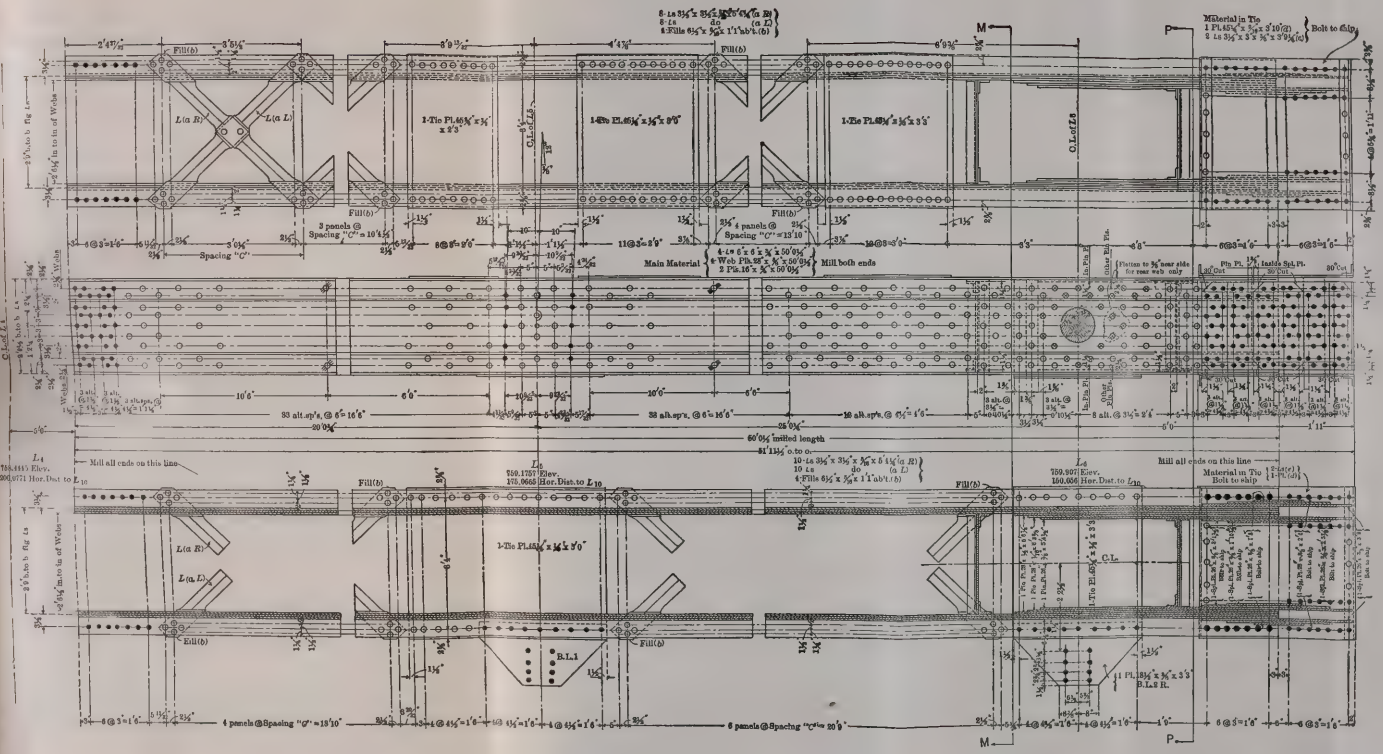
gram, Plate XII, which gives the "geometric lengths" of all members. The geometric lengths were corrected by an amount equal to the change of length of the member under full load, increasing the geometric length of compression members and shortening that of tension members. The changes of length due to full load were computed for each member with the table of stress coefficients, which are the values of the ratio, u, in the formula, $\frac{p\,u\,l}{E}$. A table was then prepared showing the geometric lengths, the increase or decrease for camber, and the finished length of each member, both in feet and decimals and in feet and inches, and the finished lengths for the stringers, which, being shorter than the finished lengths of the lower chord members, were not riveted to the floor-beams until after the lower chords were under a considerable compressive stress. For erection purposes, and to carry the locomotive crane in setting the falsework bents and floor members, the stringers were supported on shelf-angles with stiffeners proportioned for the erection loads. This table, having been made and checked carefully, assisted materially in avoiding errors in the preparation of the detailed plans.

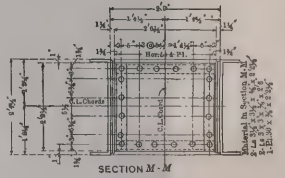
Provision for Adjustment at the Anchorage.—The lower anchor-bars, up to the first pin below the top of the masonry, were embedded in the concrete, but above this pin an open well was left to permit of some lateral movement and adjustment. After the erection was completed, it was planned to fill the well with concrete through a hole left for the purpose in the top of the pier, completely embedding the anchor-bars. The rocker links connecting the pin, L_0, in the bottom chord with the pin in the upper end of the anchor-bars were not bored until after the anchor-bars were in place. Elevations were then carefully taken on the upper pin-holes in the bars, and the rockers were bored to such lengths as were required to compensate for errors in setting the anchorages. The actual variations were small fractions of an inch.

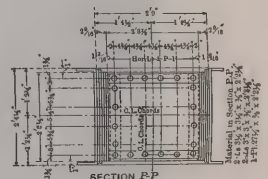
The $9\frac{3}{4}$-in. pin, engaging the upper end of the anchor-bars and the lower end of the rocker, was made with a reduced diameter of $7\frac{3}{4}$ in. at each end where it passed through the webs of the shoe. The pin-hole in the shoe was bored to a diameter of $10\frac{1}{32}$ in. There was then inserted at each end of the pin, and engaging the webs of the shoe, flanged bushings, bored with $\frac{1}{2}$ in. of eccentricity. As the maximum erection uplift, with the traveler near the center of the suspended span, was

5-98A

DETAILS OF BOTTOM CHORD
L_4–L_6


M
P
Fill(b)
L(a. R)
L(a. L)
1-Tie Pl.46⅝" x ¾" x 3'3"
1-Tie Pl.45¾" x ⅞" x 3'0"
C.L. of Ties
Material in Tie
1 Pl.45¾" x ⅞" x 3'10"(c)
2 Le 3½" x 3½" x ⅜" x 3'0½"(a)
Bolt to ship
3 panels @ Spacing "C" = 10'1½"
4 panels @ Spacing "C" = 13'10"
Main Material
4-Le 6" x 6" x ⅞" x 50'0¾"
6 Web Pls.28" x ⅞" x 50'0¾"
2 Pls.16" x ⅞" x 50'0¾"
Mill both ends
Spacing "C"
33 alt. sp'a. @ 6'= 16'6"
33 alt. sp'a. @ 6'= 16'6"
12 alt. sp'a. @ 6'= 6'0"
3 alt. @ 18¾"
5 alt. @ 3½" = 2'6"
60'0¾" limited length
61'11¾" c. to c.
10-Le 3½" x 3½" x ⅜" x 5'4½"(a. R)
10 Le " " " " (a. L)
4-Fills 6½" x ⅞" x 1'1"(a'b'c'b)
Mill all ends on this line
L_4
788.4445 Elev.
200.0771 Hor. Dist. to L_{10}
L_5
759.1757 Elev.
170.9663 Hor. Dist. to L_{10}
L_6
759.907 Elev.
150.606 Hor. Dist. to L_{10}
Material in Tie
Bolt to ship
B.L.1
B.L.2 R.
1 Pl.18¾" x ⅜" x 3'3"
4 panels @ Spacing "C" = 13'10"
6 panels @ Spacing "C" = 20'0"


SECTION M-M
Fillet to ⅜" near side for rear web only
Fin. Pls. 18" Inside Set. Pl.
Lat. Cut
90° Cut
Horiz. Lat. Pl.
L. Chords
Material in Section M-M
2-Le 6" x 6" x ⅞" x 8'0"
2-Le 6" x 6" x ⅞" x 8'0"
2-Le 3½" x 3½" x ⅜" x 8'0"
1 Pl. 18¾" x ⅜" x 8'0"



nearly equal to the maximum uplift for the finished bridge, with live load covering the cantilever arms and suspended span only, the eccentrics were adjusted and tap-bolted to the webs of the shoes when the traveler was at its extreme position. This adjustment proved to be efficient as well as extremely simple and inexpensive. It was well adapted for the conditions in this case, but would probably not be available in others.

Lower Chords.—In detailing the lower chords, an effort was made, with considerable success, to use a standard rivet spacing throughout, notwithstanding the variable panel lengths and camber increments. Plates XVI and XVII show the bottom chord members, L_4-L_6 and L_6-L_8. As may be seen, the rivet spacing, for a distance of 7 ft. 3 in. on each side of the main panel points and 15 in. and a fraction on each side of the sub-panel points, is fairly close, with thirty-three alternate spaces of 6 in. in the 25-ft. panels. (In the 37 ft. 6-in. panels there are fifty-seven alternate spaces of 6 in.) The twelve alternate spaces of 4½ in. on each side of the main panel points provided for all the lower chord splices in the cantilever and anchor arms except two.

The splices in the lower chord were designed for about 50% of the strength of the members, and consequently increased in length toward the cantilever pier, L_{10}. The rivet spacing in the splices was 1½ in., alternating, which interspaced in the 4½-in. alternate spacing without changing the stagger. This permitted a considerable similarity in the rivet spacing of the different chord members without carrying the 1½-in. spaces beyond the required length of the splice-plates at any point, and made the spacing similar in all splice- and pin-plates. As the bottom chords were parallel with the grade of the roadway, and the intermediate posts and hangers were vertical, the riveted connections at the sub-panel points had a bevel equal to the grade, requiring an odd space on each side of the sub-panel points. This space was also made to serve the purpose of taking up the variation in length of the member due to the camber increment and to the angle between the axis of the member and a horizontal line, thus concentrating all the fractional spacing at one point.

Lower Chord Lattice.—The contract plans required the lower chords to have a double lattice, at top and bottom, made of 6-in. channels, at 10½ lb. per ft., for the anchor and cantilever arms, and of 5-in. channels, at 6½ lb. per ft., for the suspended span. This was changed

to 5 by $\frac{1}{2}$-in. flat bars in the lower chord sections between L_6 and L_{14}, due to the change from four to eight angles in the main chord sections, as has been mentioned. In developing the details, it was found that the 5-in. channel was not wide enough to accommodate a sufficient number of rivets, and it was also considered that the metal of the webs was entirely too thin. The effort to find a satisfactory substitute, without increasing the weight, resulted in the use of single angles flattened at the ends where they connected with the main member and at the center where they crossed each other. Experiments showed that, when the ends were flattened out, the center of gravity of the angles was in the plane of the under side of the flattened portion.

Four full-sized comparative compression test pieces were then fabricated and tested. A $3\frac{1}{2}$ by $3\frac{1}{2}$ by $\frac{5}{16}$-in. angle was compared with a 6-in., $10\frac{1}{2}$-lb. channel, and a 3 by 3 by $\frac{5}{16}$-in. angle was compared with a 5-in., $6\frac{1}{2}$-lb. channel, the former intended for the anchor and cantilever arms and the latter for the suspended span. These test pieces were made of the exact length, from center to center of inside rivets, that the lattice bars would have in the finished bridge, and were riveted to shoes adapted to bear on the plattens of the testing machine, so as to bring the compressive stress on the test piece in the same line, as nearly as possible, as would occur in the work. Of course, these tests did not represent the conditions of stress to which lattice bars would be subjected in a member of the bridge, but they furnished comparative tests, as nearly as it was practical to make them. An excess of rivets was used in the connections, because the object was to test the sections and not the riveted connections. Table 2 gives the results of these tests.

TABLE 2.—Results of Tests of Channels and Angles.

	Ultimate Strength.		
	Total.		Per square inch, actual. in pounds.
	Computed,* in pounds.	Actual. in pounds.	
6-in., 10-lb. channel	53 600	55 500	17 950
$3\frac{1}{2}$ by $3\frac{1}{2}$ by 5/16-in. angle	47 750	47 050	22 820
5-in., $6\frac{1}{2}$-lb. channel	32 200	30 450	15 620
3 by 3 by 5/16-in. angle	38 700	46 070	25 870

* These were computed by Tetmajer's formula for the ultimate strength of small columns, as given by C. C. Schneider, Past-President, Am. Soc. C. E., in his report on the Quebec Bridge, $f = 44\,100 - 162\,\frac{l}{r}$, modified by the calculated effect of eccentricity of connections.

PLATE XVII.
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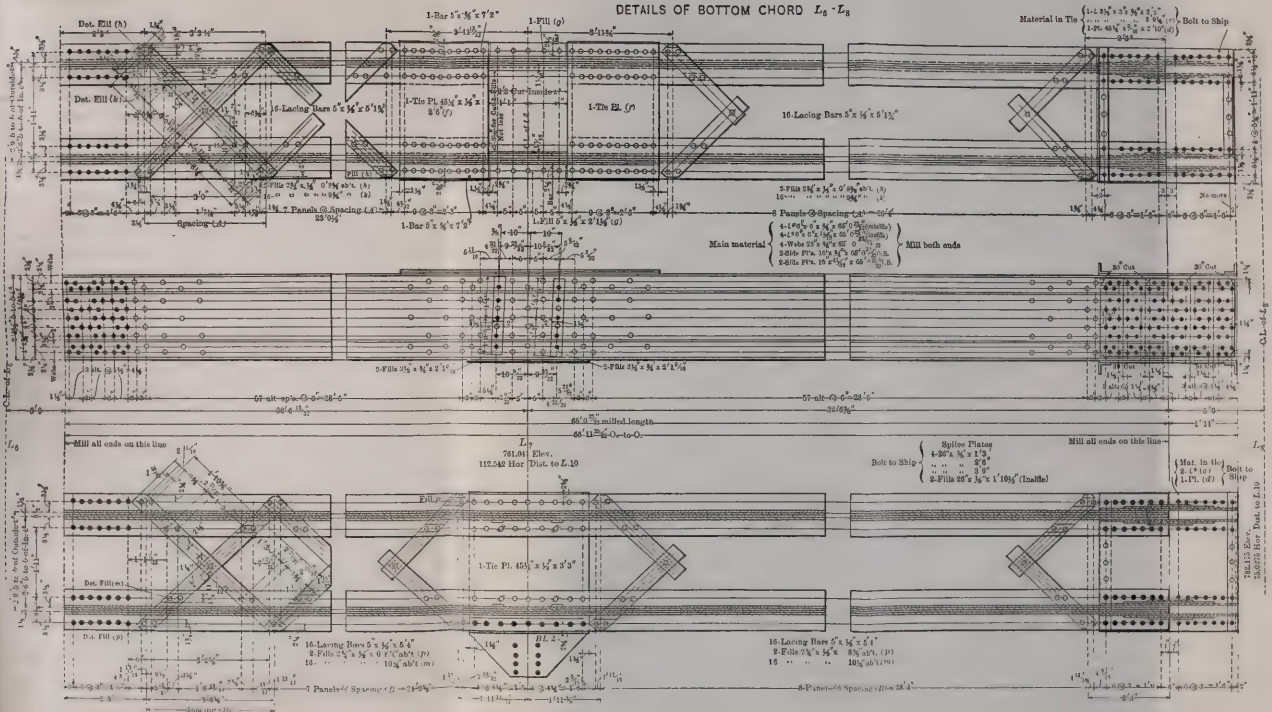
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.

DETAILS OF BOTTOM CHORD L_6-L_8


Det. Fill (h)
1-Bar 5"x⅝"x 7'2"
1-Fill (g)
Material in Tie
Bolt to Ship
Det. Fill (k)
16-Lacing Bars 5"x ⅜"x 5'1⅛"
1-Tie Pl. 45¼"x ⅜"x 2'6" (f)
Cut Inside
1-Tie Pl. (f)
16-Lacing Bars 5"x ⅜"x 5'1⅛"
2-Fills 23¼"x ⅜"x 0'8⅝" ab't. (k)
16 " " 10¾" " (a)
7 Panels @ Spacing (A) = 23'0½"
Spacing (A)
1-Bar 5"x ⅝"x 7'2"
L-Fill 5"x ⅜"x 2'1⅝" (g)
8 Panels @ Spacing (A) = 26'0"
No rivets
Main material
4-L 6"x6"x ⅞" x 65'0" (outside)
4-L 6"x6"x ⅞"x 65'0" (inside)
4-Webs 23"x ⅝"x 65'0"
2-Side Pl's. 10"x ⅝"x 65'0" (outside)
2-Side Pl's. 15"x ⅝"x 65'0" (inside)
Mill both ends
2-Fills 23½"x ⅝"x 2'1⅝"
3-Fills 23½"x ⅝"x 2'1⅝"
30° Cut
57 alt. sp's @ 0'-8" = 38'6"
57 alt. @ = 38'6"
65'0" = milled length
65'11" = C. to C.
Mill all ends on this line
L_7
761.04 Elev.
112.542 Hor. Dist. to L 10
Spl'co Plates
Bolt to Ship
4-20"x ⅜"x 1'8"
" " " 2'6"
2-Fills 20"x ⅜"x 1'10½" (Inside)
Mat. in tie
2-L's (e)
1-Pl. (d)
Bolt to Ship
Det. Fill (m)
Det. Fill (p)
1-Tie Pl. 45½"x ½"x 3'3"
Fill
16-Lacing Bars 5"x ⅜"x 5'4"
2-Fills 23¼"x ⅜"x 0'8⅝" ab't. (p)
16 " " 10¾" ab't. (m)
16-Lacing Bars 5"x ⅜"x 5'4"
2-Fills 23¼"x ⅜"x 8⅝" ab't. (p)
16 " " 10¾" ab't. (m)
7 Panels @ Spacing = 23'0½"
8 Panels @ Spacing = 26'4"
Spacing (B)
761.15 Elev.
338.05 Hor. Dist. to L 10

These tests indicate an efficiency for the angles of from 30 to 50% greater than for the channels, and the computations show that this is due to the very small eccentricity of the angle connections.

The deflections were recorded and the load removed with each increment of 5 000 lb. up to 30 000 lb., at which point the 6-in. channel showed a permanent set, whereas the 3½-in. angle did not. At 40 000 lb. the 6-in. channel showed a flexure 25% greater than the 3½-in. angle. The permanent set of the 5-in. channel and the 3-in. angle was not obtained. The flexure was always in the direction indicated by the slight eccentricity of the riveted connections, except for the 3-in. angle, which began to deflect in the opposite direction; but, after the load reached 40 000 lb., the eccentricity of the connection caused the flexure to reverse. Between the shops and the testing laboratory, this angle had been bent in handling or shipping; therefore it was a defective test piece at the start. The test, however, proved that this initial bend was in the direction which neutralized the effect of eccentricity until the load reached 40 000 lb., and it is probable that the ultimate load was greater than would have been the case if the angle had been straight. All these test pieces failed by column flexure, the back of the channels and the root of the angles being concave. On the basis of these tests, the angle sections were approved and used instead of the channels in the lower chords from L_0 to L_6, and from L_{14} to L_{22}. They afforded ample room for the riveted connections, and the saving in weight fully compensated for the additional cost of fabrication. It also avoided cutting and splicing at intersections, which is an objection to the channel lattice bar.

Shoes and Connections of Floor-Beams and Sidewalk Brackets at L_{10}.—The shoes at L_{10}, which carry the entire load that comes on the cantilever piers, are built up of plates and angles with three ribs and three diaphragms, and take their loads through 14-in. pins. They have a base of 8 by 9 ft., and a height, to center of pin, of 8 ft. 6⅛ in. The thickness of the bearing of the three ribs is 17¾ in., and the shipping weight is 37 860 lb. The shoes are connected across the pier by box struts, 4 ft. deep and 2 ft. 6 in. wide, latticed on four sides and riveted to the shoes. The end connection plates of these struts are extended up to form a seat for the floor-beam at L_{10}, bringing the end reactions of this beam directly to the shoes. This detail was adopted in order to avoid the almost impossible field connections of the floor-beam to the tower columns, as contemplated by the contract plans. The brackets support-

ing the sidewalk at the tower, L_{10}, were also supported directly from the shoes for the same reasons.

Forked Connections at Main Panel Points Common to Two Diagonal Compression Members.—To meet problems of packing, and to facilitate erection, the sub-diagonals, M_7-L_8 and L_8-M_9, were connected at L_8 on common pin-plates which formed a fork. This was shop-riveted to L_8-M_9 with field-riveted connections for M_7-L_8, 3 ft. 6 in. from the pin center. The same detail was used at L_{12} and L_{18}. The detail at L_{10}, for the members, U_8-M_9-L_{10} and L_{10}-M_{11}-U_{12}, is similar, except that the fork is a separate member with field-riveted connections for both of the main members.

This detail introduced some secondary stresses, which were duly considered and provided for. In this case these were neither excessive nor difficult to handle, but, with heavier members, the secondary stresses might be objectionably large.

Trussing in Center Cross Panels of Suspended Span.—The contract plans called for cross diagonal eye-bars, without adjustment, in the center panels of the suspended span, U_{22}-M_{23}-L_{22}. As in all other parts of the bridge the lower sub-diagonals were compression members, and as it was necessary to introduce adjustment in these panels, the upper sub-diagonals, U_{22}-M_{23}, were made of adjustable eye-bars connecting on the pin, M_{23}, outside of all the other members. The lower sub-diagonals on the north side (the first side erected) carried shop-riveted pin-plates at M_{23}, which plates were provided with field-riveted connections for L_{22}-M_{23} (south side), U_{23}-M_{23}, and M_{23}-L_{23}. This member was erected on the north side, and was held in position by the adjustable diagonals, U'_{23}-M_{23}, engaging the pin, M_{23}. In closing the bridge, all the other members connecting on M_{23} came together automatically and without any difficulty, except that required to guide them into the connection plates. As soon as the suspended span was swung, the upper adjustable diagonals on the south side were slipped over the ends of the pins and the nuts were then screwed on, completing the connection.

Connection of End Post to Top Chord of Suspended Span at U_{18}.—The end posts of the suspended span have, shop-riveted to their upper ends at U_{18}, and integral with them, a short section or stub end of the upper chord. The main chord section is field-riveted to this piece 3 ft. 7 in. forward of the pin, U_{18}. The short section of upper chord is extended back of the panel point 5 ft. 3 in. to take the subsidiary pin to

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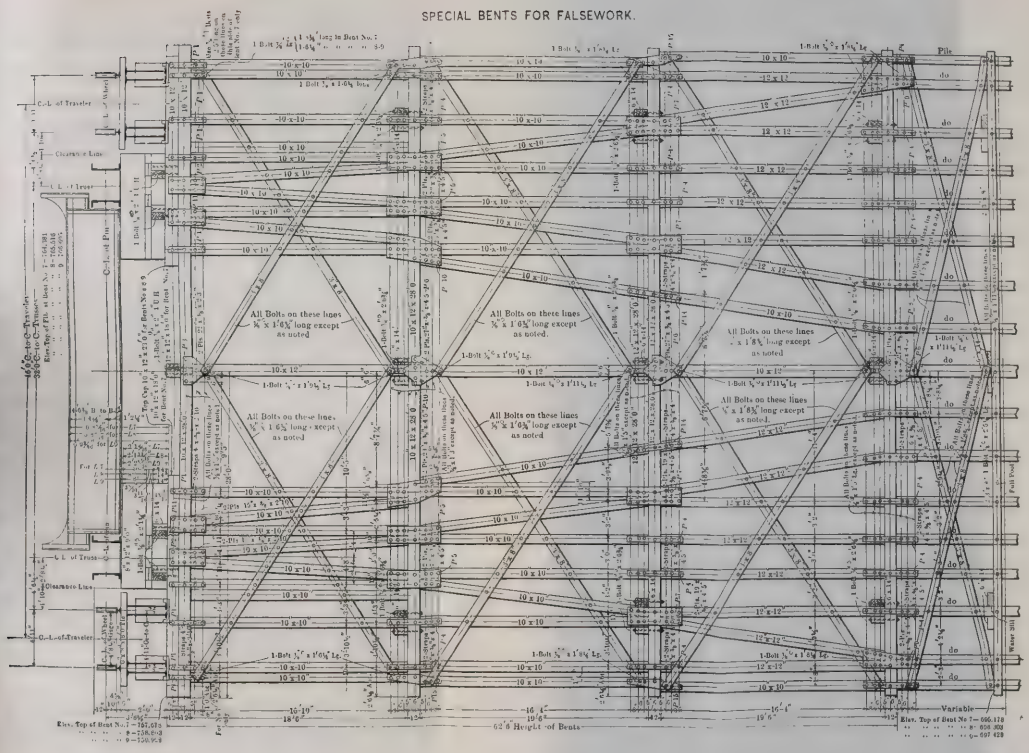
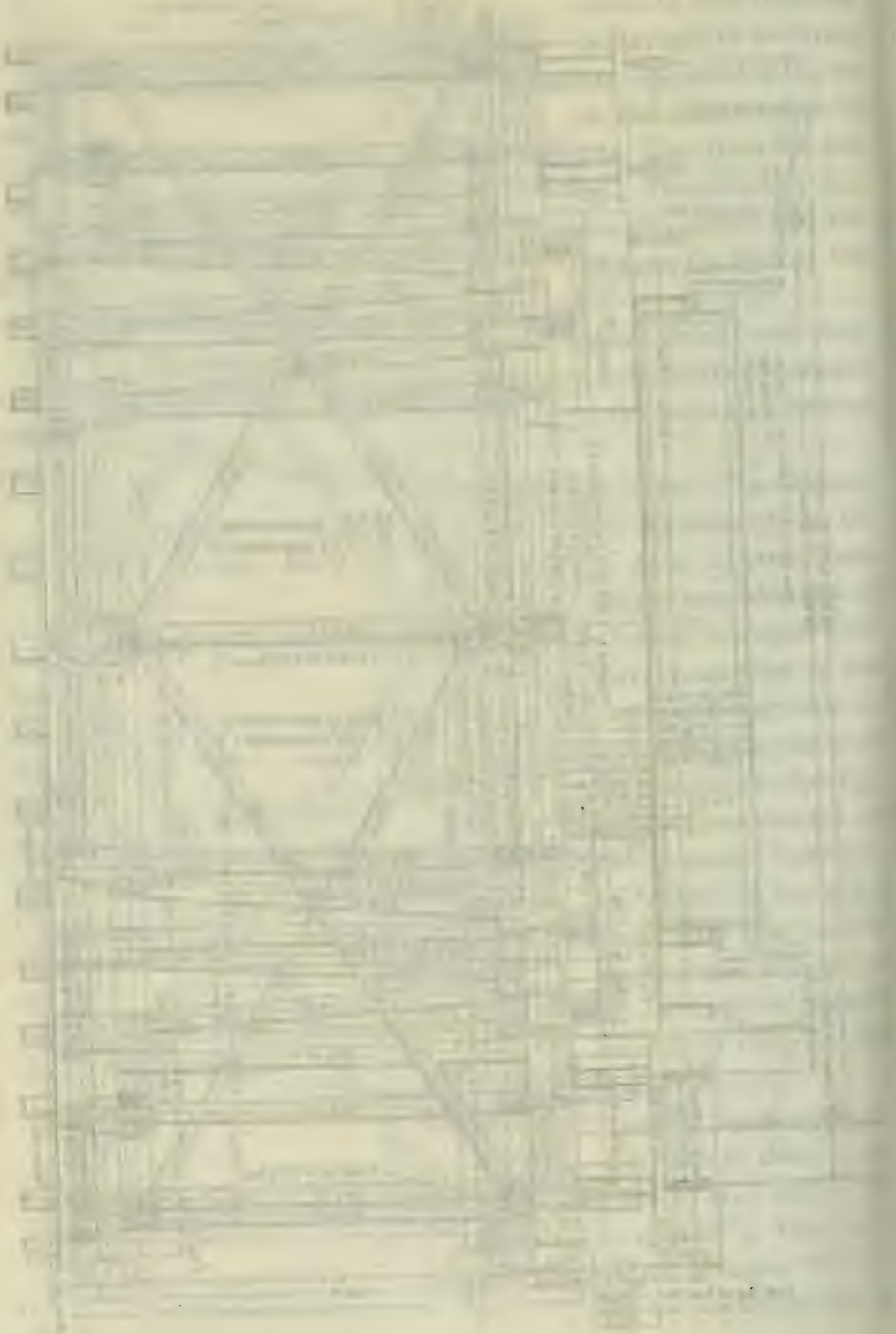
PLATE XVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1255.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.

SPECIAL BENTS FOR FALSEWORK.


All Bolts on these lines ⅞" x 1 6/8" long except as noted.
All Bolts on these lines ⅞" x 1 6/8" long except as noted
All Bolts on these lines ⅞" x 1 6/8" long except as noted
All Bolts on these lines ⅞" x 1 6/8" long except as noted
All Bolts on these lines ⅞" x 1 6/8" long except as noted
All Bolts on these lines ⅞" x 1 6/8" long except as noted
62'6 Height of Bents
Pile
do
Variable



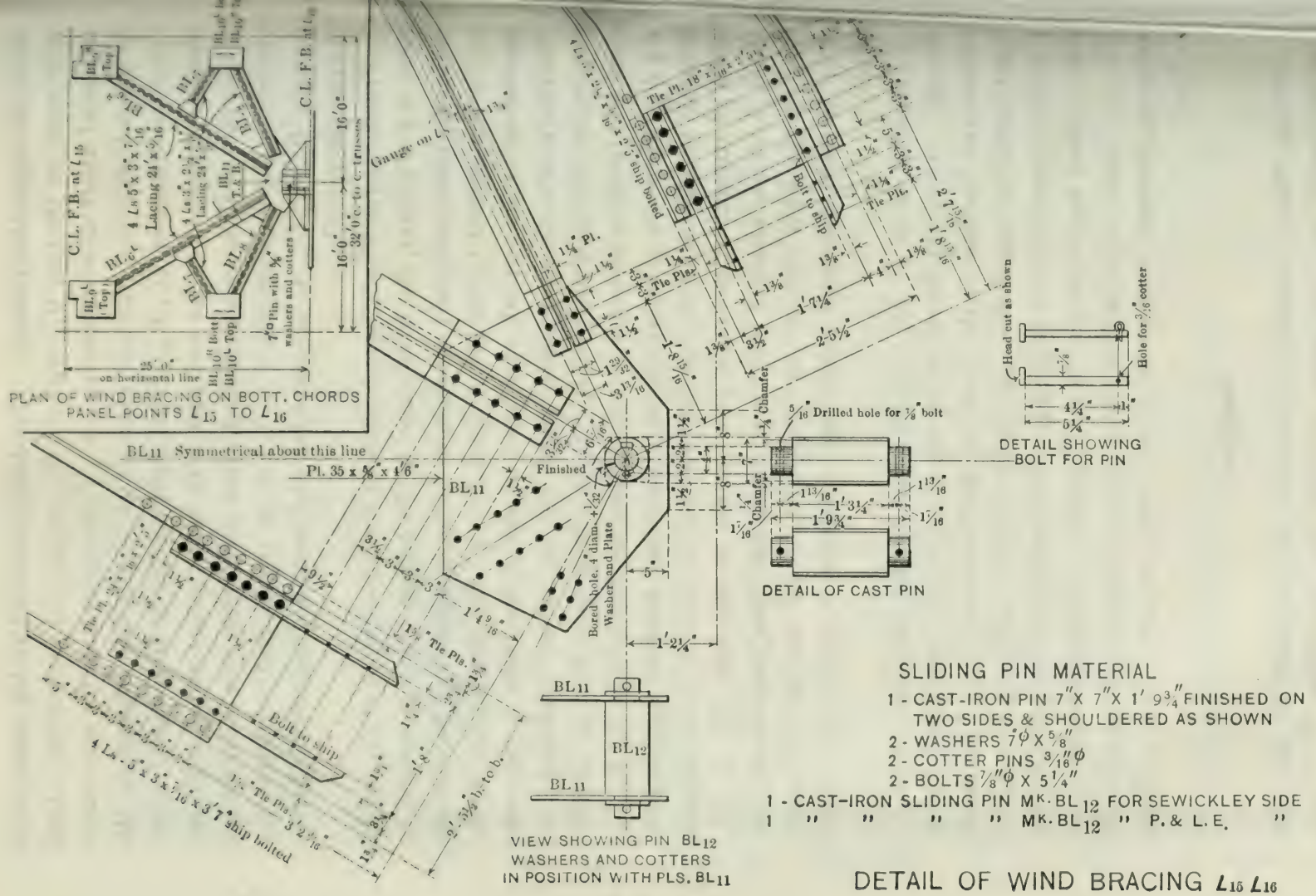
which the toggle bars connect. This detail permitted all the connections to be made on the pin, U_{18}, without the necessity of supporting a long section of upper chord projecting out. It also permitted the traveler to be erected with its principal reaction at U_{18} before connecting the upper chord section, U_{18}-U_{20}. This short section or stub end of upper chord is connected to the end post with heavy gusset-plates, sufficient to take care of all secondary stresses which occurred at this point during erection. The separate pin, U_{18} A, for the toggle bars, permitted the triangle, U_{16}-L_{16}-U_{18}, to be connected and made self-supporting before making the pin, U_{18}.

Provisions for Wind Shear in Bottom Frame.—Except at L_{16}, where there is a slip joint, the wind shear in the bottom frame is carried by the buckle-plates and their connections to the stringers and floor-beams, and is transmitted to the lower chord through the connections of the floor-beams to the vertical truss members and of the latter to the lower chord by pins at the main panel points and by riveted diaphragm connections at the sub-panel points. These connections were all investigated and found to be ample, except at the sub-panel points, L_7, L_9, L_{11}, and L_{13}, where the sub-verticals were somewhat deficient in transverse strength to transmit the load safely; but, as there is a considerable excess of strength at the main panel points on each side (L_6, L_8, L_{10}, and L_{12}), it was assumed that the stringers and their connections, particularly the lines adjacent to the trusses, would take up the excess shear at these sub-panel points and distribute it to the main panel points.

The wind shear at L_{16} is transmitted to the lower chords at L_{15} through a triangular frame which has its apex in a cross-head sliding between two brackets at the center of the floor-beam, L_{16}. This detail is not novel, but was worked out independently, before the description of a similar one had been published. The legs of the triangle are composed of four angles latticed in an I section. They are braced laterally, by a frame, to the bottom chord or erection strut just back of the hinge pin, serving to brace the latter during erection. This is shown in Fig. 6.

Erection.

General Plan.—Naturally, the erection of steel began at each side with the riveted pony truss spans, of which there were three, 75 ft. long,


PLAN OF WIND BRACING ON BOTT. CHORDS
PANEL POINTS L_{15} TO L_{16}
C.L. F.B. at L_{15}
C.L. F.B. at L_{16}
4 Ls 5" x 3" x 7/16" Lacing 21' x 9/16
4 Ls 3" x 2½" x 7/16" Lacing 24' x 7/16
16'-0"
16'-0"
32'-0" c. to c. Trusses
7" Pin with ¾" washers and cotters
25'-0" on horizontal line
BL10" Bott
BL10" Top
Gauge on L
Tie Pl. 18" x 7/16" x 2'-3½"
Bolt to ship
Tie Pls.
1¼" Pl.
Tie Pl.
Head cut as shown
Hole for 3/16" cotter
DETAIL SHOWING BOLT FOR PIN
5/16" Drilled hole for ⅞" bolt
¼" Chamfer
DETAIL OF CAST PIN
BL11 Symmetrical about this line
Pl. 35 x ⅞ x 4'6"
Finished
Bored hole, 4" diam.
Washer and Plate
Tie Pl. 24" x 7/16"
Bolt to ship
4 Ls - 5" x 3" x 7/16" x 3'7" ship bolted
VIEW SHOWING PIN BL12 WASHERS AND COTTERS IN POSITION WITH PLS. BL11
BL11
BL12
BL11

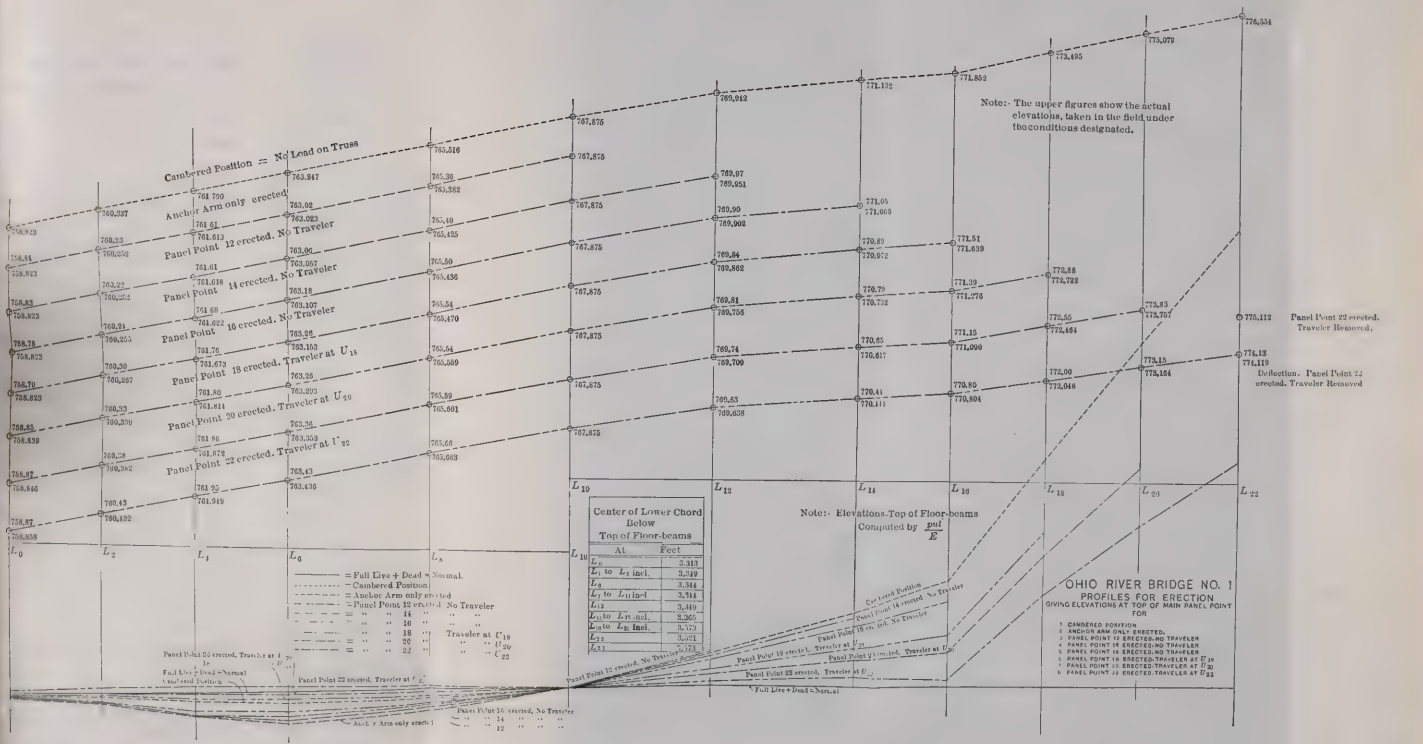
SLIDING PIN MATERIAL

1 - CAST-IRON PIN 7" X 7" X 1' 9¾" FINISHED ON TWO SIDES & SHOULDERED AS SHOWN
2 - WASHERS 7" ϕ X ⅝"
2 - COTTER PINS 3/16" ϕ
2 - BOLTS ⅞" ϕ X 5¼"
1 - CAST-IRON SLIDING PIN MK. BL_{12} FOR SEWICKLEY SIDE
1 " " " " " MK. BL_{12} " P. & L. E. "

DETAIL OF WIND BRACING L_{15} L_{16}

FIG. 6.

604A


Note:- The upper figures show the actual elevations, taken in the field under the conditions designated.
Note:- Elevations-Top of Floor-beams Computed by $\frac{pul}{E}$
Center of Lower Chord Below Top of Floor-beams
At | Feet
L_0 | 3.313
L_1 to L_2 incl. | 3.349
L_3 | 3.344
L_4 to L_{11} incl. | 3.344
L_{12} | 3.349
L_{13} to L_{21} incl. | 3.365
L_{22} to $L_{\bar{2}\bar{2}}$ incl. | 3.373
$L_{\bar{2}\bar{2}}$ | 3.321
$L_{\bar{2}\bar{1}}$ | 3.373
Cambered Position = No Load on Truss
Anchor Arm only erected
Panel Point 12 erected, No Traveler
Panel Point 14 erected, No Traveler
Panel Point 16 erected, No Traveler
Panel Point 18 erected, Traveler at U_{18}
Panel Point 20 erected, Traveler at U_{20}
Panel Point 22 erected, Traveler at U_{22}
Panel Point 22 erected. Traveler Removed.
Deflection. Panel Point 22 erected. Traveler Removed
= Full Live + Dead = Normal.
= Cambered Position
= Anchor Arm only erected
= Panel Point 12 erected. No Traveler
= " " 14 " " "
= " " 16 " " "
= " " 18 " Traveler at U_{18}
= " " 20 " " " U_{20}
= " " 22 " " " U_{22}
OHIO RIVER BRIDGE NO. 1
PROFILES FOR ERECTION
GIVING ELEVATIONS AT TOP OF MAIN PANEL POINT FOR
1 CAMBERED POSITION
2 ANCHOR ARM ONLY ERECTED.
3 PANEL POINT 12 ERECTED-NO TRAVELER
4 PANEL POINT 14 ERECTED-NO TRAVELER
5 PANEL POINT 16 ERECTED-NO TRAVELER
6 PANEL POINT 18 ERECTED-TRAVELER AT U_{18}
7 PANEL POINT 20 ERECTED-TRAVELER AT U_{20}
8 PANEL POINT 22 ERECTED-TRAVELER AT U_{22}

on the Sewickley or north side, and two of 75 ft. with one of 118 ft. over the Pittsburgh and Lake Erie tracks, on the south side. These trusses were assembled on the ground and lifted into place with the 30-ton locomotive crane. In setting the 118-ft. trusses, the locomotive crane was assisted by a boom derrick.

Falsework.—The falsework for the anchor arms was assembled in single stories and lifted into place by the locomotive crane, the piles having already been driven and cut off level. Plate XVIII shows the bents under panel points 7, 8, and 9, and Fig. 7 shows the provision for supporting the stringers and locomotive crane at panel point 16. As soon as the upper story on each bent was completed, the necessary blocking was arranged and the floor-beams and stringers were set thereon in final position and bolted up. The buckle-plates were then placed and bolted, and on them the track was laid for the advance of the locomotive crane. This operation was repeated, bent by bent, until the falsework reached the cantilever pier or one or two panels beyond. The wing or gantry traveler was then erected on the falsework, stock I-beams being used to support the traveler tracks on each side, as shown in the plans. As the erection of the steelwork for the anchor arm trusses proceeded, the locomotive crane continued to set the falsework bents and the floor-beams and stringers, out to panel point 18.

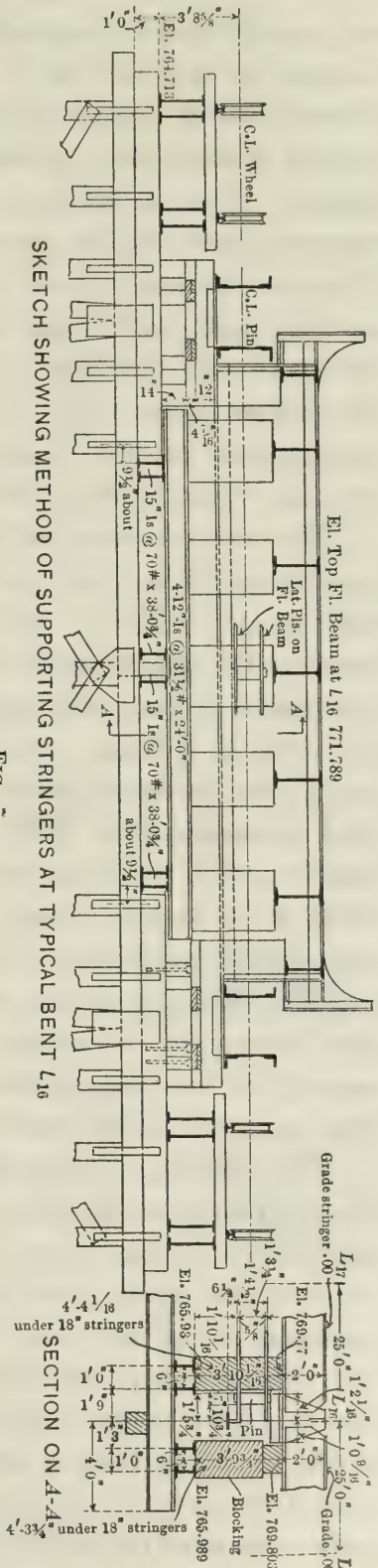


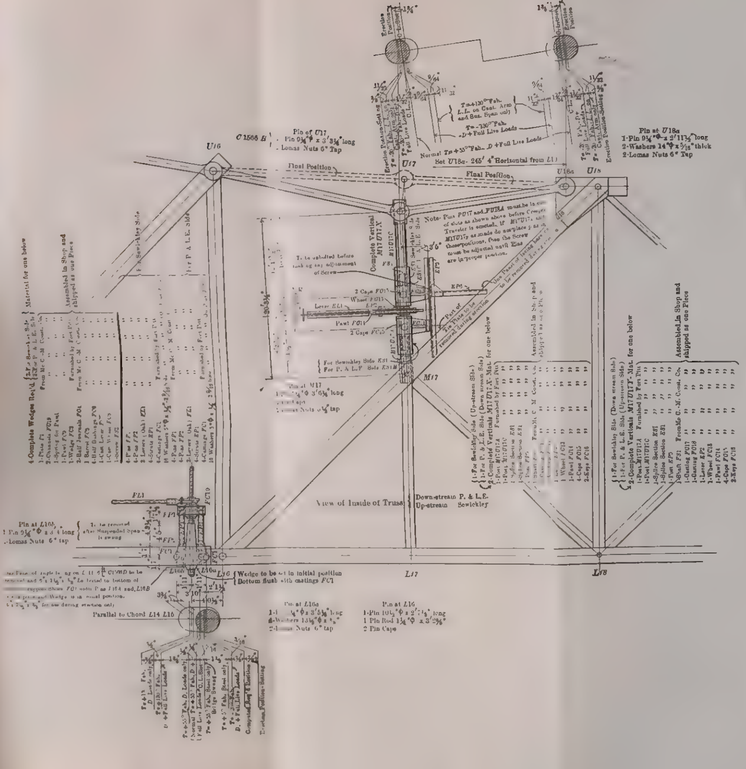
FIG. 7.

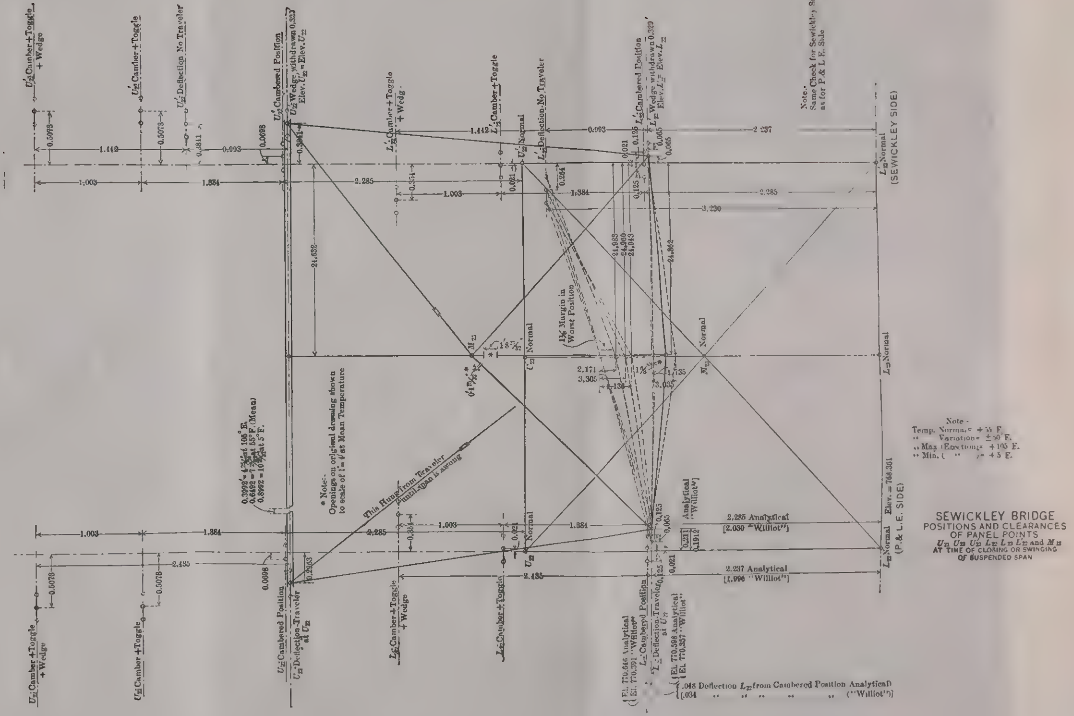
The locomotive crane set the falsework bents, floor-beams, stringers etc., quickly and economically, fully justifying the trouble and expense incurred to provide for it. The heavy reactions, which often came almost entirely on two wheels of one truck, were difficult to take care of, and at one time it appeared to be impracticable to make use of this method. That a solution was finally reached is largely due to the resourcefulness of the erection department of the Bridge Company. The extra counterbalance was removed from the rear end of the locomotive crane, and special outriggers were provided which distributed the loads over four lines of stringers. The stringer reactions at the floor-beams were taken by shelf-angles with supporting stiffeners designed for erection loads. These have been referred to under the heading, "Camber and Finished Lengths of Truss Members."

The original intention had been to erect the cantilever arm, from panel point 12 out, with a creeper traveler, and some provisions were made to do so; but, as the restrictions on account of navigation only required the maintenance of a clear waterway of 500 ft., it was found permissible to carry the falsework out to panel point 18 on one side of the river at a time. After all the falsework was removed from the north side, it was replaced, out to panel point 18, on the south side, thus preserving the 500-ft. opening at all times. This method of erection, with the wing traveler on falsework out to the second panel point of the suspended span (18) was very economical and expeditious and avoided considerable difficulty and expense in carrying the creeper traveler down the eye-bar chain of the cantilever arm. It also was of great assistance in making the panel points 16, 17, and 18, which, on account of the large number of members necessary to hold up at one time, were among the most difficult to close.

This plan of erecting the cantilever arm with falsework required careful provision, in order to avoid any possibility of a concentrated reaction occurring at any of the bents outside of the cantilever piers, which might easily have been caused by an unequal settlement of the falsework bents, and would have crushed the bents or buckled some of the truss members, particularly in the sub-panels. If the cantilever arm had been carried entirely by the falsework during its erection it would have also been a difficult and hazardous operation to swing without dangerously over-loading some of the bents and truss members as the computed deflection of panel point 18, with the erection compl

606A.


View of Inside of Truss
Down-stream P. & L.E.
Up-stream Sewickley
Float Position
Final Position
U16
U17
U18
L16
L17
L18
M17


Note -
Temp. Normal = +55 F.
Variation = ±50 F.
Max. (Erection) = +105 F.
Min. (") = +5 F.
SEWICKLEY BRIDGE
POSITIONS AND CLEARANCES
OF PANEL POINTS
AT TIME OF CLOSING OR SWINGING
OF SUSPENDED SPAN

to that point, was about 0.8 ft. With panel point 22 erected, the computed deflection of panel point 18 was about 1.45 ft. Each truss was fully adequate to carry its own weight as a cantilever, together with the weight of the traveler at L_{22}, which, of course, they had to do in erecting the suspended span. Therefore, as each successive panel was erected by the gantry traveler, from panel point 10 to panel point 18, the wedges were backed out at all points outside of the cantilever pier so as to swing the span entirely. With every load added to the trusses great care was exercised to make sure that the wedges were not too tight, but they were kept in place for lateral stiffness only and to hold the falsework in line with the steel superstructure. From this it will be seen that the falsework under the cantilever arm, after the locomotive crane had set the floor system, never carried anything except the gantry traveler and the panel of steelwork being erected at any time, but not yet connected.

As indicated in the preceding description, the erection was carried forward from panel point 18 by the true cantilever method, with a cantilever or creeper traveler moving on the upper chords of the suspended span. This traveler was erected by the wing traveler at panel points 16, 17, and 18, and was supported over the toggle bars by special I-beam tracks.

As shown by Plate XII, the roadway grade on the anchor arms is 3% ascending, which was introduced in order to get the head-room under the channel span required by the Government regulations. The grade from panel point 13 outward is 0.8%, with a vertical curve between panel points 10 and 13. The wheel base of the traveler was 75 ft., with three trusses, and this required special arrangements to get it over the vertical curve without overloading the stringers of the traveler track or the falsework bents. This condition was mitigated by using in the traveler track a flatter and longer vertical curve than that of the roadway, and by arranging the longitudinal sway rods in the traveler so that by slackening one pair of rods the traveler would deflect and partly adjust itself to the curve. The traveler was sufficiently strong in all details to carry its entire weight on one pair of wheels at each side, but, to have made the stringers and bents safe for such a concentrated load would have cost too much, and the danger of derailment would have been increased. The deflection of the traveler in the direction indicated was somewhat restrained by the longitudinal

struts, but a computation showed that the latter were not sufficiently stiff to cause undue concentrations when the sway rods were slack. These struts, however, proved to be so much stiffer than anticipated that they were used for staging in driving pins, etc., in preference to the staging brackets which had been provided.

The clear space between the top of rail of traveler track and bottom of traveler sill was 22 in. The largest pin that had to be driven through this space was 14 in. in diameter. This only allowed from 4 to 6 in. of clearance at the top and bottom to provide for variations in elevation and settlement of bents, camber blocking, etc. Several inches of this clearance were taken up by the adjustment of the vertical curves in the traveler track.

Much consideration was given to the hazard due to floods and other causes which might have destroyed the falsework. One of the provisions made for this was for swinging the anchor arm before the cantilever arm was erected. Erection ties were inserted, from U_2 to M_3 and from U_4 to M_5, and were removed as the shear in these panels was reversed, and the lower sub-diagonals were reinforced to meet this condition; but it was recognized that the greatest danger concerned the bents under the cantilever arms. These bents, 11 to 18, inclusive, did not carry any part of the steelwork after it had been fully connected, and therefore the hazard was practically confined to the possible loss of the falsework, provided only that the warning should afford sufficient time to run the traveler back to panel points 12, 13, and 14, lash it to the trusses, knock out the blocking, and let the falsework go. For floods and ice there would be sufficient warning for this. The trusses were more than able to carry this load except during the short time when panel point 18 had been made and the creeper traveler erected. With the two travelers on the cantilever arm at one time, there would have been some overloading, but this critical period was confined to the short time between the erection of the creeper traveler and the dismantling of the wing or gantry traveler. The work was planned so that this critical period was not encountered during ordinary times of high water. Fortunately, however, it did not become necessary to resort to any of these expedients.

Erection Deflections, Changes of Lengths, and Adjustments.—The deflections and elevations at the top of the floor-beams were computed analytically for the conditions of no load, for four conditions of loading in the finished bridge, and for seven conditions of erection loading.

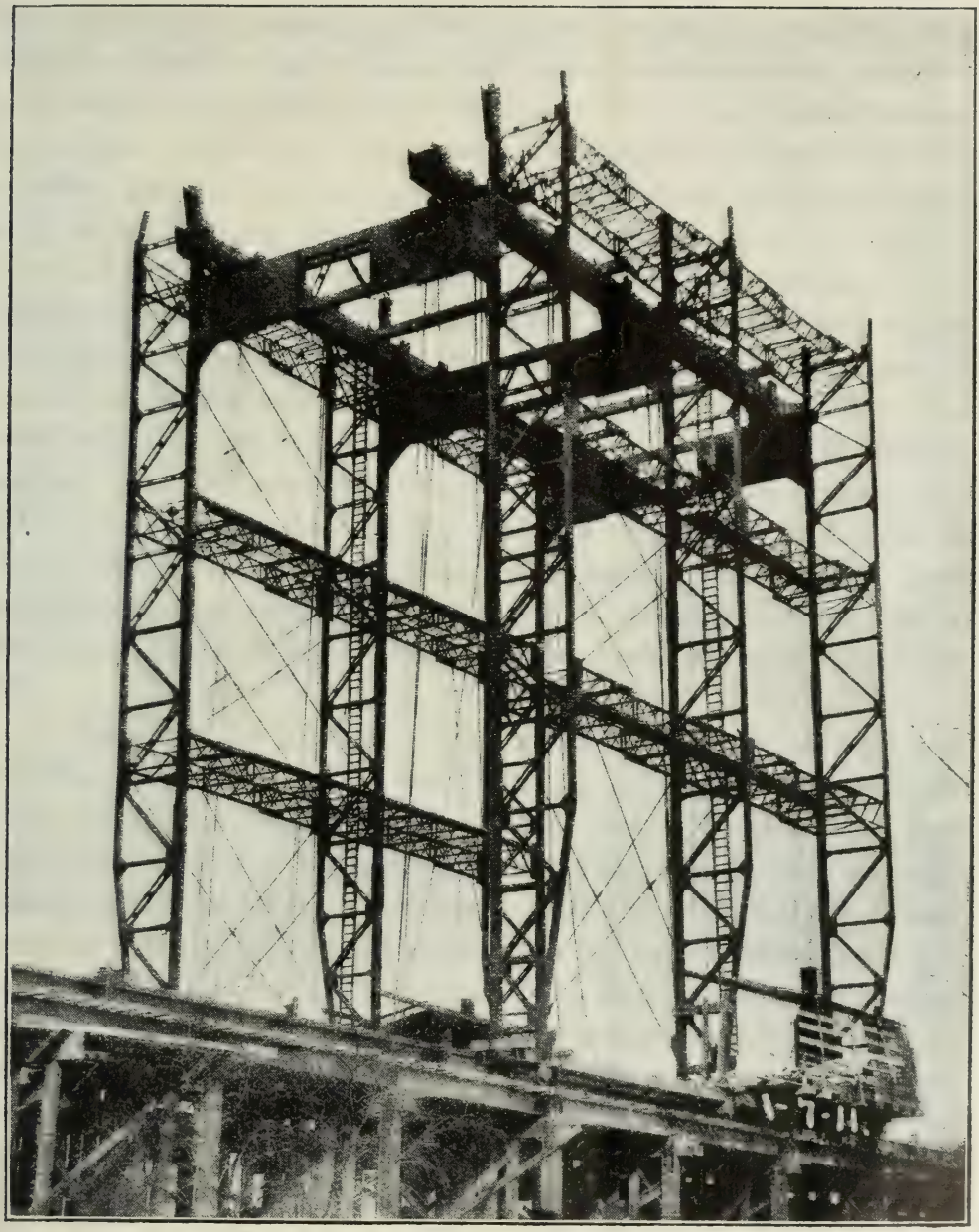


FIG. 8.—THE WING TRAVELER, SEWICKLEY BRIDGE.

Plate XIX shows the deflection profiles for erection conditions. Williot graphical diagrams for displacements were also made for five conditions, the vertical co-ordinates being compared with and checked by the computed deflections. Plate XIX shows the elevations taken on the floor-beams at different stages during the erection. The variations between the computed and actual deflections are generally quite small, and many of the differences would be very much less had it not been for the fact that some of the material, particularly for sidewalks and railings, was not erected in position but piled on the bridge at other points, whereas the computations considered all material as erected in final position, with no other material on the bridge excepting the traveler. The variations between the computed and actual deflections are less than might be expected when the differences in loading conditions are considered. The data from the Williot diagrams and from the computed deflections, Table 3, added to thermal changes, gave the minimum clearance or play required between U_{16} and U_{18} and between L_{15} and L_{16}, and also the run required for both toggles and wedges. On account of its extreme importance, these computations were made five times, by three different methods and by three men, independently. The provisions made are shown on Plate XX.

At this point the question of toggles and wedges was taken up and carried so far as to determine the dimensions required for the principal parts and make a preliminary estimate of cost. Inquiries were then made as to whether or not the toggle and wedge apparatus from some previous cantilever bridge was available, with the result that the Bridge Company obtained the use of the set which had just done service in the Beaver Bridge. It had been designed for a load several times greater than the Sewickley Bridge, and required some special provisions to make it available, but there was the satisfaction of knowing that it had been tested under a much greater load than it would have to carry in this bridge. It is essentially the same as all the toggle and wedge devices that have been used in cantilever erections for the past ten years.

On May 15th, 1911, the lower chord of the suspended span was closed by driving the pins, L_{23}. The distance between U_{22} and U'_{22} was then measured and found to agree very closely with the computed clearance. The top chord was closed and the suspended span was swung on May 16th, 1911. During the entire work of erection, nothing of consequence

TABLE 3.—Displacements as Determined by Williot Diagrams, with Parallel Columns Showing Comparison, or Check, of Vertical Displacements with Values Computed by $\frac{pul}{E}$.

Point.	Cambered Position. Displacement. Horizontal.	Vertical. Williot.	Vertical. $\frac{pul}{E}$	Corrected Co-ordinates, Origin at L_{10}. Horizontal.	Vertical.	Corrected elevation above datum.
L_{10}	0	0	0	0	0	764.531
L_8	— 0.0275	— 0.1060	— 0.1087	— 75.0275	— 2.3560	762.175
L_6	— 0.0560	— 0.1240	— 0.1282	— 150.0560	— 4.6240	759.907
L_4	— 0.0771	— 0.0865	— 0.0907	— 200.0771	— 6.0865	758.4445
L_2	— 0.0950	— 0.0400	— 0.0431	— 250.0950	— 7.5400	756.991
L_0	— 0.1030	— 0.0208 = $\frac{1}{4}$"(adjusted)		— 300.1030	— 9.0208	755.5102
U_{10}	— 0.3464	+ 0.0583	.....	— 0.3464	+ 114.7253	879.2563
U_8	— 0.1740	— 0.1300	.....	— 75.1740	+ 77.7320	842.2630
U_6	— 0.1422	— 0.1012	.....	— 150.1422	+ 53.9508	818.4818
U_4	— 0.1286	— 0.0684	.....	— 200.1286	+ 45.9016	810.4326
U_2	— 0.1205	— 0.0270	.....	— 250.1205	+ 42.4730	807.0040
U_0	— 0.2265	— 0.0920	.....	— 37.7265	+ 94.5490	859.0800
M_9	— 0.0862	— 0.0655	.....	— 37.5862	+ 38.8655	803.3965
L_9	.....	.....	.....	— 37.5138	— 1.1780	763.3530
U_7	— 0.1520	— 0.1352	.....	— 112.6520	+ 64.1948	828.7258
M_7	— 0.1150	— 0.1264	.....	— 112.6150	+ 36.5546	801.0856
L_7	.....	.....	.....	— 112.542	— 3.490	761.041
U_5	.....	.....	.....	— 175.1354	+ 49.9262	814.4572
M_5	.....	.....	.....	— 175.1078	+ 23.9326	788.4636
L_5	.....	.....	.....	— 175.0665	— 5.3553	759.1757
U_3	.....	.....	.....	— 225.1246	+ 44.1873	808.7183
M_3	.....	.....	.....	— 225.1112	+ 19.1811	783.7121
L_3	.....	.....	.....	— 225.0861	— 6.8132	757.7178
M_1	.....	.....	.....	— 275.1114	+ 16.7269	781.2579
L_1	.....	.....	.....	— 275.0990	— 8.2804	756.2506
L_{12}	+ 0.0223	+ 0.3534	+ 0.3600	+ 75.0223	+ 2.0554	766.5864
L_{14}	+ 0.0488	+ 0.7797	+ 0.7980	+ 150.0488	+ 3.2177	767.7487
L_{16}	— 0.2065	+ 1.0995	+ 1.1184	+ 199.7935	+ 3.7295	768.2605
U_{10}	— 0.3464	+ 0.0583	.....	— 0.3464	+ 114.7253	879.2563
U_{12}	— 0.3024	+ 0.3298	.....	+ 74.6976	+ 82.0318	846.5628
U_{14}	— 0.2235	+ 0.7840	.....	+ 149.7765	+ 61.1540	825.6850
U_{16}	— 0.2065	+ 1.0761	.....	+ 199.7935	+ 53.6901	818.2211
M_{10}	— 0.1480	+ 0.0247	.....	— 0.1480	+ 49.0247	813.5557
U_{11}	— 0.3440	+ 0.1427	.....	+ 37.1560	+ 96.8757	861.4067
M_{11}	— 0.1520	+ 0.1620	.....	+ 37.3480	+ 41.0130	805.5440
L_{11}	.....	.....	.....	+ 37.5115	+ 1.164	765.695
U_{13}	— 0.2560	+ 0.5504	.....	+ 112.2440	+ 70.1314	834.6624
M_{13}	— 0.1262	+ 0.5545	.....	+ 112.3738	+ 42.6245	807.1555
L_{13}	.....	.....	.....	+ 112.5359	+ 2.7029	767.2339
U_{15}	.....	.....	.....	+ 174.7865	+ 57.4176	821.9486
M_{15}	.....	.....	.....	+ 174.9224	+ 28.4506	792.9816
L_{15}	.....	.....	.....	+ 175.0567	+ 3.5759	768.1069
Dead Load Only. Bridge Swung.						
L_{10}	.....	0	0	0	0	764.531
L_8	.....	+ 0.0817	+ 0.0832	.....	.....	762.2567
L_6	.....	+ 0.0970	+ 0.1007	.....	.....	760.004
L_4	.....	+ 0.0583	+ 0.0614	.....	.....	758.5028
L_2	.....	+ 0.0290	+ 0.0297	.....	.....	757.0200
L_0	+ 0.0750	+ 0.0205	+ 0.0205	.....	.....	755.5307
U_{10}	+ 0.2710	— 0.043	.....	.....	.....	879.2133
U_2	+ 0.0976	+ 0.0179	.....	.....	.....	807.0219
L_{12}	.....	— 0.2816	— 0.2863	.....	.....	766.3048
L_{14}	.....	— 0.6130	— 0.6181	.....	.....	767.1357
L_{16}	+ 0.1690	— 0.8615	— 0.8590	.....	.....	767.3990
U_{16}	+ 0.1690	— 0.8443	.....	.....	.....	817.3768

TABLE 3.—(*Continued.*)

LIVE LOAD ON ANCHOR ARM ONLY.

Point.	Displacement. Horizontal.	Displacement. Vertical. Williot.	Displacement. Vertical. $\frac{pul}{E}$	Corrected Co-ordinates, Origin at L_{10}. Horizontal.	Corrected Co-ordinates, Origin at L_{10}. Vertical.	Corrected elevation above datum.
L_{10}	0	0	0	0	0	764.531
L_8	.....	+ 0.0235	+ 0.0234	.....	.....	762.1985
L_6	.....	+ 0.0075	+ 0.0077	.....	.....	759.9145
L_4	.....	— 0.0255	— 0.0251	.....	.....	758.4190
L_2	.....	— 0.0396	— 0.0392	.....	.....	756.9514
L_0	.....	0	0	....	.....	755.5102
U_{10}	+ 0.2130	— 0.0428	.....	.....	.....	879.2135
L_{12}	.....	— 0.2368	— 0.2374	.....	.....	766.3496
L_{14}	.....	— 0.5280	— 0.5342	.....	.....	767.2207
L_{16}	+ 0.1362	— 0.7460	— 0.7661	.....	.....	767.5145
U_{16}	+ 0.1362	— 0.7288	.....	.....	.....	817.4923
LIVE LOAD ON CANTILEVER ARM AND SUSPENDED SPAN ONLY.						
L_{10}	0	0	0	0	0	764.531
L_8	.....	+ 0.1580	+ 0.1596	.....	.....	762.3330
L_6	.....	+ 0.2030	+ 0.2048	.....	.....	760.1100
L_4	.....	+ 0.1667	+ 0.1686	.....	.....	758.6112
L_2	.....	+ 0.0970	+ 0.0972	.....	.....	757.0880
L_0	.....	+ 0.0350	+ 0.0350	.....	.....	755.5452
U_{10}	+ 0.3990	— 0.056	.....	.....	.....	879.2003
L_{12}	.....	— 0.3892	— 0.3917	.....	.....	766.1972
L_{14}	.....	— 0.8505	— 0.8522	.....	.....	766.8982
L_{16}	+ 0.2322	— 1.1800	— 1.2021	.....	.....	767.0805
U_{16}	+ 0.2322	— 1.1566	.....	.....	.....	817.0645
ANCHOR ARM ONLY SWUNG.						
L_{10}	0	0	0	0	0	764.531
L_8	.....	— 0.1342	— 0.1341	.....	.....	762.0408
L_6	.....	— 0.2234	— 0.2244	.....	.....	759.6836
L_4	.....	— 0.1745	— 0.1767	.....	.....	758.2700
L_2	.....	— 0.0851	— 0.0853	.....	.....	756.9059
L_0	.....	0	0	.....	.....	755.5102
U_{10}	— 0.1339*	— 0.0010*	....	—0.4794′ =—5¾*	+114.7243*	879.2553*
U_8	— 0.0772	— 0.1100	.....	— 75.2512	77.6220	842.1530
U_2	+ 0.0028	— 0.0782	.....	—250.1177	42.3948	806.9258
L_{12}	.....	.....	.....	+ 75.0205*	+ 2.1417*	766.6727*
U_{12}	.....	.....	.....	+ 74.6032*	+ 82.1174*	846.6484*

*Computed from Williot co-ordinates of U_8. Members between $U_8 = L_{10} = U_{12}$ assumed at finished cambered lengths without stress.

occurred which had not been foreseen and provided for. There were no losses of either men or material, and no serious injuries were reported. One man fell into the water, but was rescued without serious results.

General Directions for Erection, and Order of Closing Operations Issued to the Erection Department on May 19th, 1910.—The spaces for pilot nuts, L_{18}, L_8 to L_{12}, etc., are somewhat scant. They had better be started off before the pin is entirely home, that is, $\frac{3}{8}$ in. back of the final position, or just flush.

The bottom chord splices of the anchor arm must be bolted and drifted before the anchor arm is swung.

The expansion pockets for stringers on beam L_9 are not heavy enough to take the weight of the locomotive crane, and the stringers must be blocked up from the pier.

The eccentric and pin at P'_1 must be well greased and covered with canvas to prevent rusting before the adjustment is completed.

Fully bolt and drift the top chord splices at U_{18} before advancing the creeper traveler beyond that point.

The locomotive crane must not be run out past the tower, L_{19}, after the creeper traveler is erected.

While the traveler is at U'_{22} (north side), and before moving it, connect the counters, U'_{22}-M_{23}, and the lower sub-diagonals, M_{23}-L'_{22} (which have shop-riveted gusset-plates at M_{23}), on pin M_{23}.

Be sure that the counters, U'_{22}-M_{23} (north side), are screwed up to just the right length, and that they are not too short in any case. If they are $\frac{1}{16}$ to $\frac{1}{8}$ in. too long, it will be better than to have them too short, because, if too short, L_{22}-M_{23} (south side) cannot be connected in closing.

With the traveler at U_{22} (south side), make the pin, L_{23}, engaging L_{22}-L_{23}, L_{23}-L'_{22}, and M_{23}-L_{23}.

The lower sub-vertical, M_{23}-L_{23}, having a field-riveted connection at M_{23}, will have to be tied up or supported until the span is swung.

Bring U'_{22} level with U_{22} by backing out the wedges on the north side at L'_{16}.

Set the upper sub-vertical, U_{23}-M_{23}, bolting it to the gussets at M_{23}.

Set the top chord, U'_{22}-U_{22}, bolting up the splice to U'_{22}-U'_{20} and to the top of U_{23}-M_{23}. It will then be self-supporting, its half-length, U_{23}-U_{22}, projecting toward the south side as a cantilever.

Set and bolt the top lateral struts, U_{23}-U_{23}, and the top laterals, U'_{22}-U_{23}.

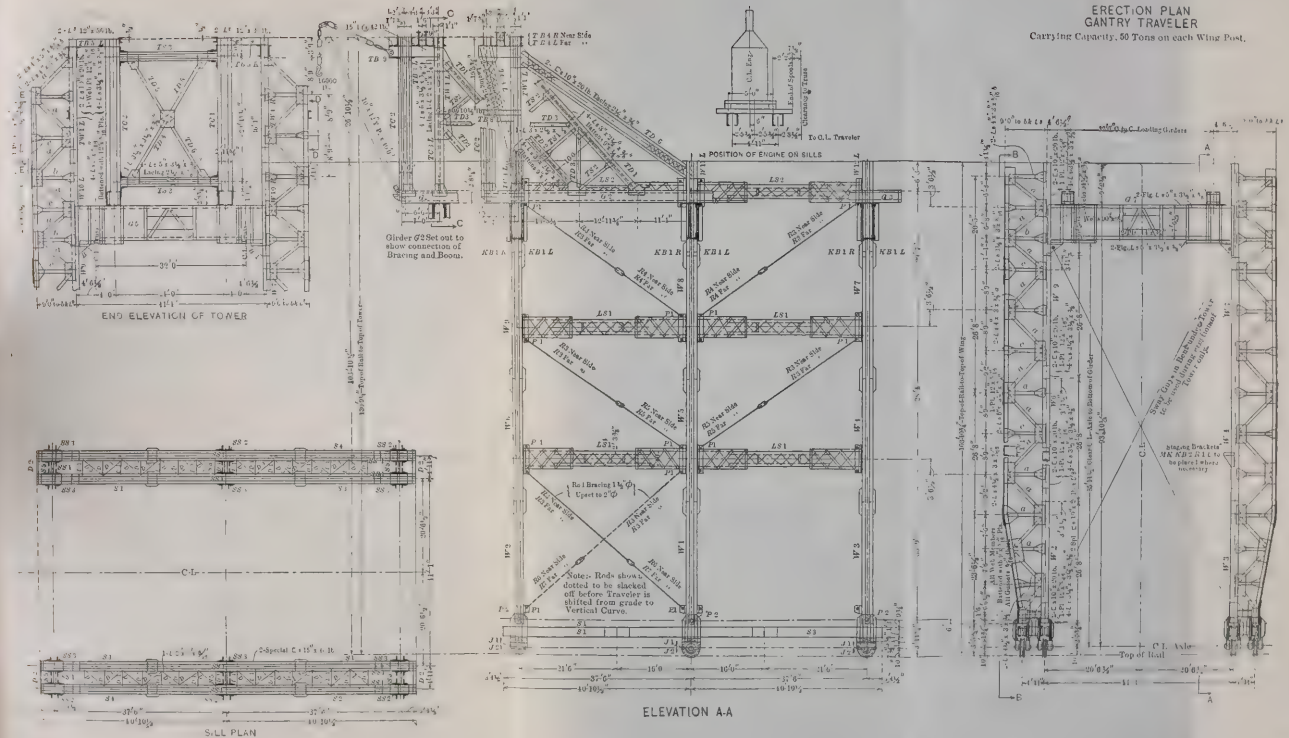
Release the toggles until the top chord, U'_{22}-U_{22}, is in bearing at U_{22}, and bolt the splice.

The top laterals, U_{22}-U_{23}, must be put in place as soon as possible after the top chords, U_{22}-U'_{22}, are set, and before the toggles are fully released. When fully released, the span is changed from cantilever to span with both ends supported, and the conditions of the lateral frame change. With the traveler on top, this is serious, and it is very important to place the laterals as directed above.

After the top chord, U_{22}-U'_{22}, is set, and the center panel top laterals are in place, and the top chord splices are bolted (with toggles partly released), it is very important to continue slacking off the toggle screws until the toggle bars are in a straight line (*i. e.*, entirely slacked off), before leaving the bridge, because, in the position above described, a drop in the temperature would cause the top chords to act as suspension cables and probably cause a disaster.

PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1255.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.

ERECTION PLAN
GANTRY TRAVELER
Carrying Capacity, 50 Tons on each Wing Post.


END ELEVATION OF TOWER
Girder G2 Set out to show connection of Bracing and Boom.
POSITION OF ENGINE ON SILLS
To C.L. Traveler
Note:- Rods shown dotted to be slacked off before Traveler is shifted from grade to Vertical Curve.
ELEVATION A-A
SILL PLAN
C.L. Axle
Top of Rail

The operation of slacking off the toggle screws may continue uninterruptedly after the top chord is connected.

The wedges should then be entirely slacked off before bolting M_{23}-L_{23} on the gusset at M_{23}.

After the span is swung, it will be necessary to lift the weight of the members on the pin, L_{23}, to take the sag out of the lower chord at this point before L_{23}-M_{23} can be bolted at M_{23}.

The counters, U_{22}-M_{23} (south side), can be slacked with turnbuckles and slipped over the ends of the pin, M_{23}, as these counters are outside (next to the pin nuts).

NOTE.—Prime (′) refers to the north side.

The Wing or Gantry Traveler.—An investigation having shown the impracticability of rebuilding the steel traveler which the Bridge Company had used during the previous year, so as to adapt it for the Sewickley Bridge, the following question was proposed: "Is it practicable to design a traveler adapted for the erection of spans of variable dimensions and weights?" Before the prices of materials and labor had advanced to their present values, and when good bridge timber was plentiful and cheap, it was common practice to build a special timber traveler for each job; but, in recent years, it has been found more economical to build travelers of steel, and they have generally been made without a sufficient range of adjustment to fit widely varying cases, although, in several instances, they have been designed with some adjustable members.

The foregoing question was answered by the proposal that, "a steel traveler is a tool or machine which should be provided with such adjustments as to make it available for the erection of all spans within a comprehensive range." A little preliminary work demonstrated the fact that it was entirely practicable to construct such a tool, and the result is the traveler first used on the Sewickley Bridge. The Bridge Company had a considerable quantity of stock which it was desired to use for the travelers. Some of this stock was perfectly good material, but of unusual sections rarely called for, and some of it, while good enough for a traveler, would hardly be acceptable for contract work. It was also desired that the material from the old steel traveler should be used in the new one, as far as available. These considerations required a number of unusual adaptations of details, but did not affect the general design materially.

Plate XXI shows the general elevations of this traveler, together with the extension on top, designed for the erection of the

towers. Fig. 12 shows the top view and plan of loading girders with and without tower. Plate XXII shows the details of the adjustable longitudinal struts.

The shoes which carry the boxes of the main wheel journals and the pins connecting the lower ends of the wings can be adjusted for panel lengths varying by 6 in. For the Sewickley Bridge, it was bolted up for panels 37 ft. 6 in. long. The maximum reach is two 40-ft., and the minimum two 24-ft., panels. For two 32-ft. panels, one of the 16-ft. sections of sill would be removed and the center shoe shifted to the center of the remaining 16-ft. section. For two 24-ft. panels, the other 16-ft. section would be removed. The projection of the sills beyond the ends is thus kept within 8 ft. 3 in. The traveler may be used with two trusses or wings instead of three.

The wings were designed so that when bolted up in a total length of about 100 ft., and lying down flat, they can be supported on one point at their center. This provision was made so that they might be tripped up from the bottom pin with a tackle made fast to one point.

The longitudinal spacing in the wings is uniformly 5 in. from end to end, with $2\frac{1}{2}$ in. edge distance at the ends of the sections, and bolts are used instead of rivets throughout all sections where the transverse girders might come. This permits the height from traveler rail to transverse girders to be adjusted to any distance up to about 95 ft., by variations of 5 in. The capacity of the traveler is sufficient to permit the addition of other sections of wings, so that the clear height could be extended up to about 125 ft.

The transverse box girders have open lattice in their center 10-ft. panels and solid-plate webs in the end panels. The latter are engaged by another set of web-plates with separate flange-angles, arranged to bolt on the outside of the center section. The holes in both sets of web-plates are punched, so that they can be bolted in any position to give clear widths of from 30 ft. 3 in. to 49 ft. 3 in. between the traveler wings. For the Sewickley Bridge, they were set at 40 ft. 3 in. clear. It is intended that these adjustments shall be made at the shop for each job, before shipping the traveler.

The loading girders were designed to give a large moment of inertia sideways. Their make-up was determined by the stock on hand. Open holes were provided in the webs to fix the saddles and prevent them from sliding longitudinally when drifting a load. Open holes were also

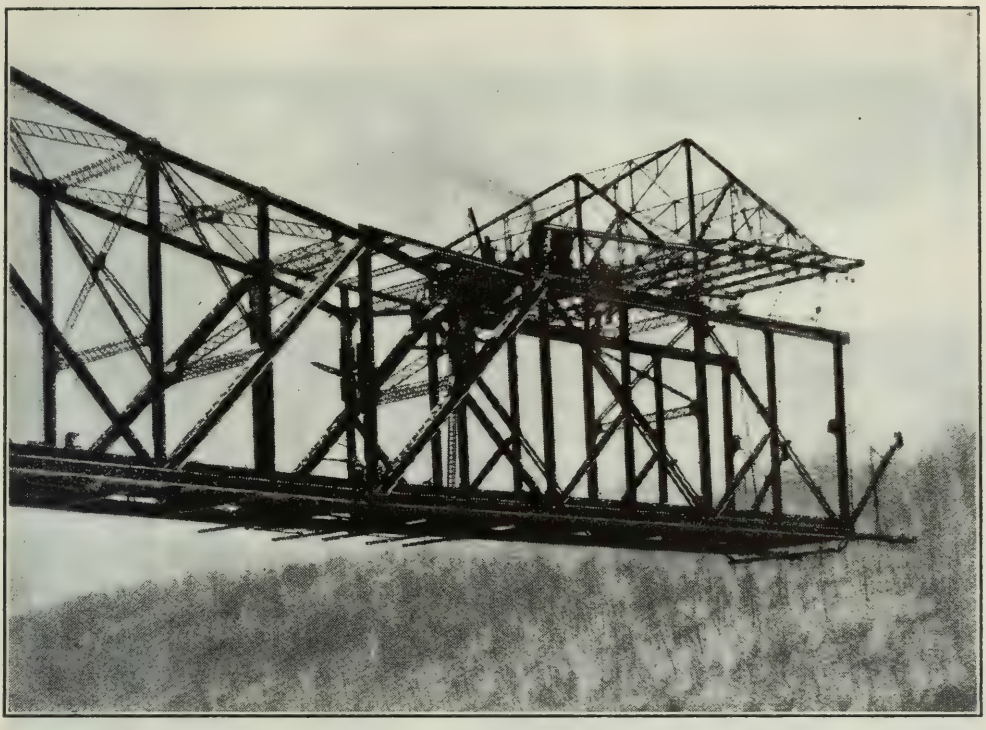


FIG. 9.—CREEPER TRAVELER AT U_{20}, NORTH SIDE.

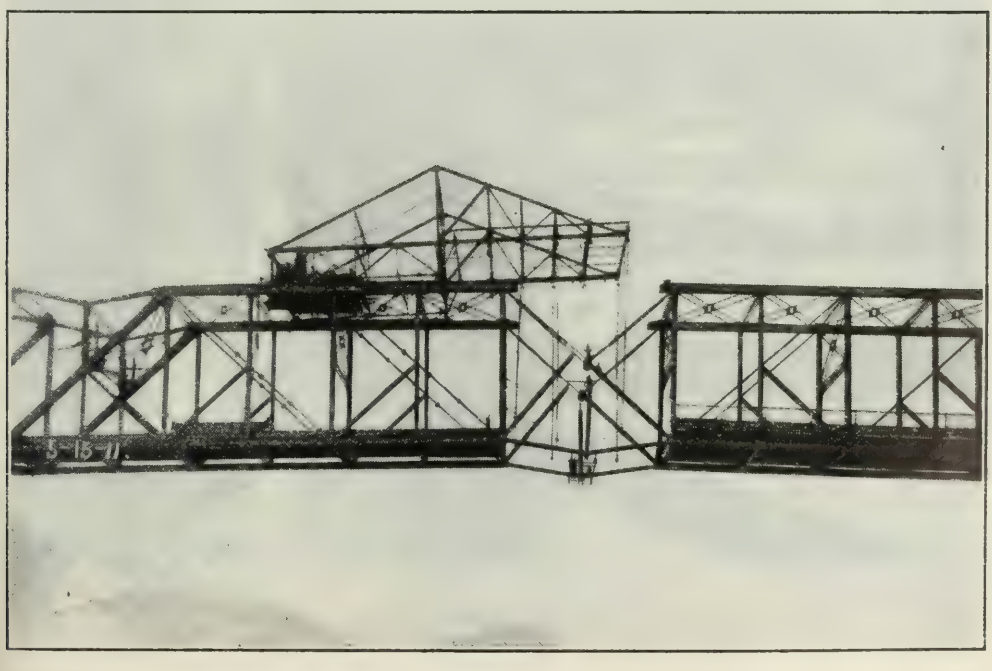


FIG. 10.—PINS AT L_{23} DRIVEN. CREEPER TRAVELER IN ADVANCE OF PANEL POINT 21.

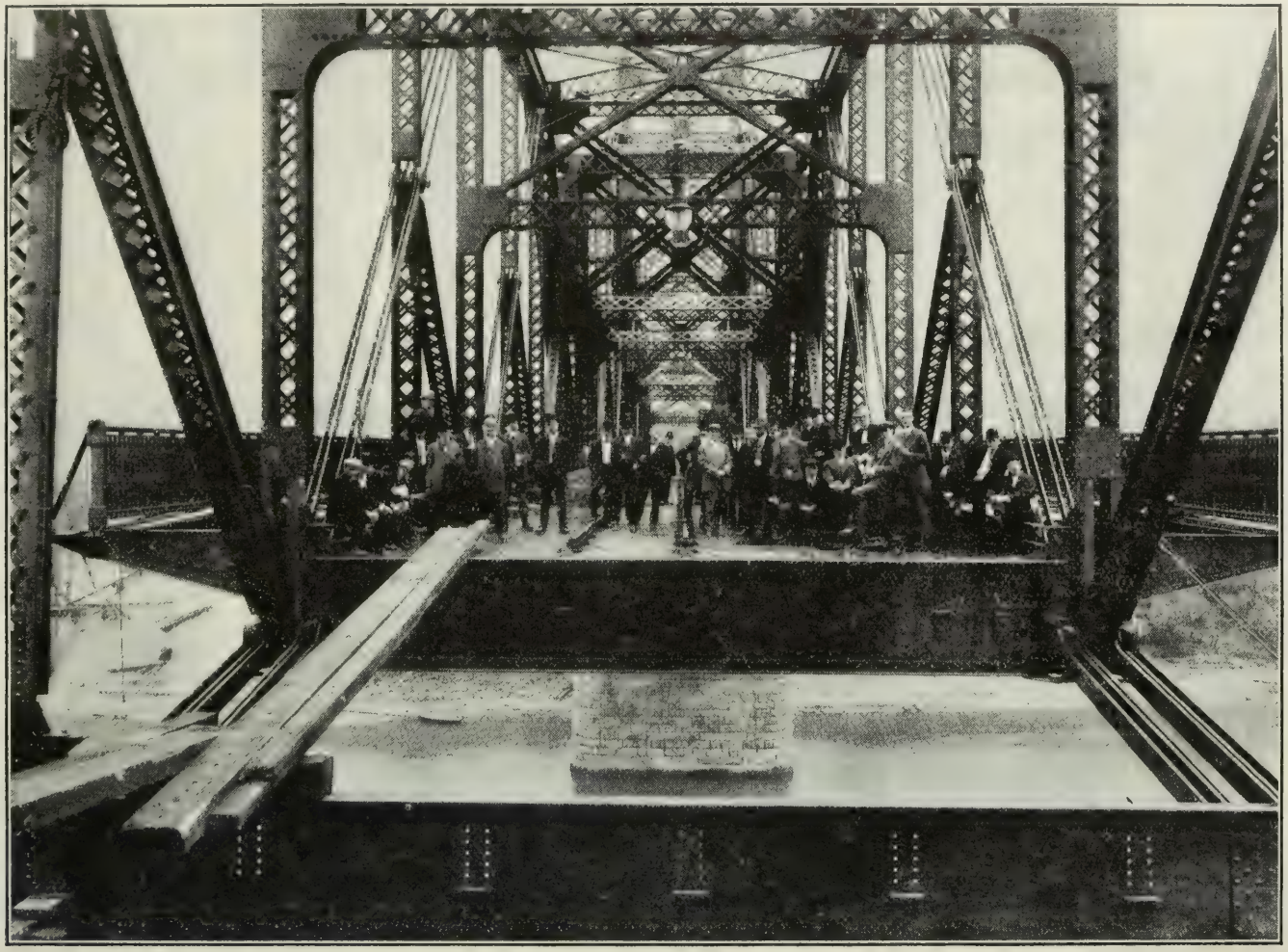
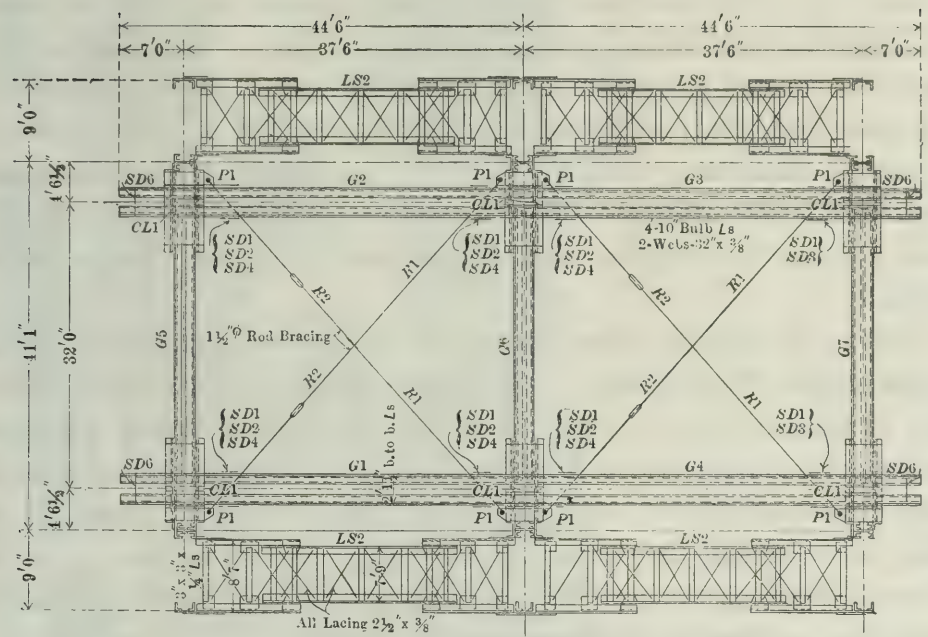
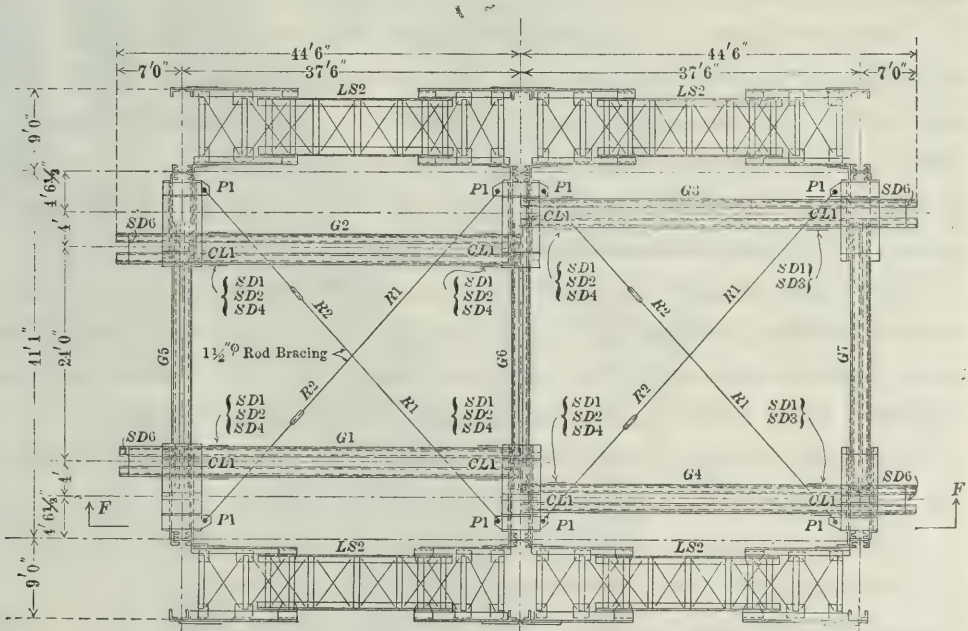


FIG. 11.—CROSS-SECTION OF SEWICKLEY BRIDGE.



TOP VIEW OF TRAVELER (WITHOUT TOWER)



PLAN OF LOADING GIRDERS. TOWER ERECTED.

GANTRY TRAVELER

FIG. 12.

provided in the end diaphragms for bolting on brackets to carry Chicago boom seats. These booms were used to set the top lateral system between U_8 and U_{12}, after the traveler had moved forward, and also to assist in supporting one end of the upper chord, U_{10}-U_{11}. The loading girders were riveted up complete at the shop, but it proved so difficult to handle them that, before taking down the traveler on the north side, the rivets connecting them through the diaphragms were cut out and bolts were substituted when they were set up again on the south side. This reduced the weight necessary to handle at one time to about one-half.

The most difficult adjustment to work out was that of the longitudinal struts (Plate XXII). The center section of these struts, which is 2 ft. 3⅜ in. deep and 7 ft. 9 in. wide, back to back of angles, telescopes into the two end sections, which are 3 ft. deep and 8 ft. 7 in. wide, back to back of angles, eight clip angles, 3½ by 3½ by ⅜ in. and 13½ in. long, at each end, providing a good long connection and 1 in. of clearance between the sections. In making this adjustment, which will generally be done at the shops, the clip angles will be removed and replaced in the required positions.

The only parts of this traveler which are not adjustable for spans of different dimensions are the 1½-in. round lateral and longitudinal sway-rods. It was considered preferable to avoid complications incident to introducing adjustments of wide range in these rods because it is very simple to make new pieces for the short ends, and the discarded pieces will be good stock which can be readily reworked.

The eight saddles for the main hoists were designed so as to travel on the loading girders with four wheels on two axles, and two of them were thus constructed. The other six were constructed with cast shoes and two 3½-in. pins to transmit the load to the girders. These pins are identical with the axles of the two fitted with wheels, so that they can be used either with wheels or with stationary shoes.

The twelve main traveler wheels were made with cast-iron centers or treads and double steel-plate flanges, riveted through, and with 7-in. journals, calculated to carry the maximum load when lifting. These wheels gave good service and no trouble. The two Mundy hoisting engines, one on each side, had three drums and four spools, and were set up on the sills in the forward bay, the longitudinal sway-rods being omitted in this bay. It was intended to use blocking between the sills

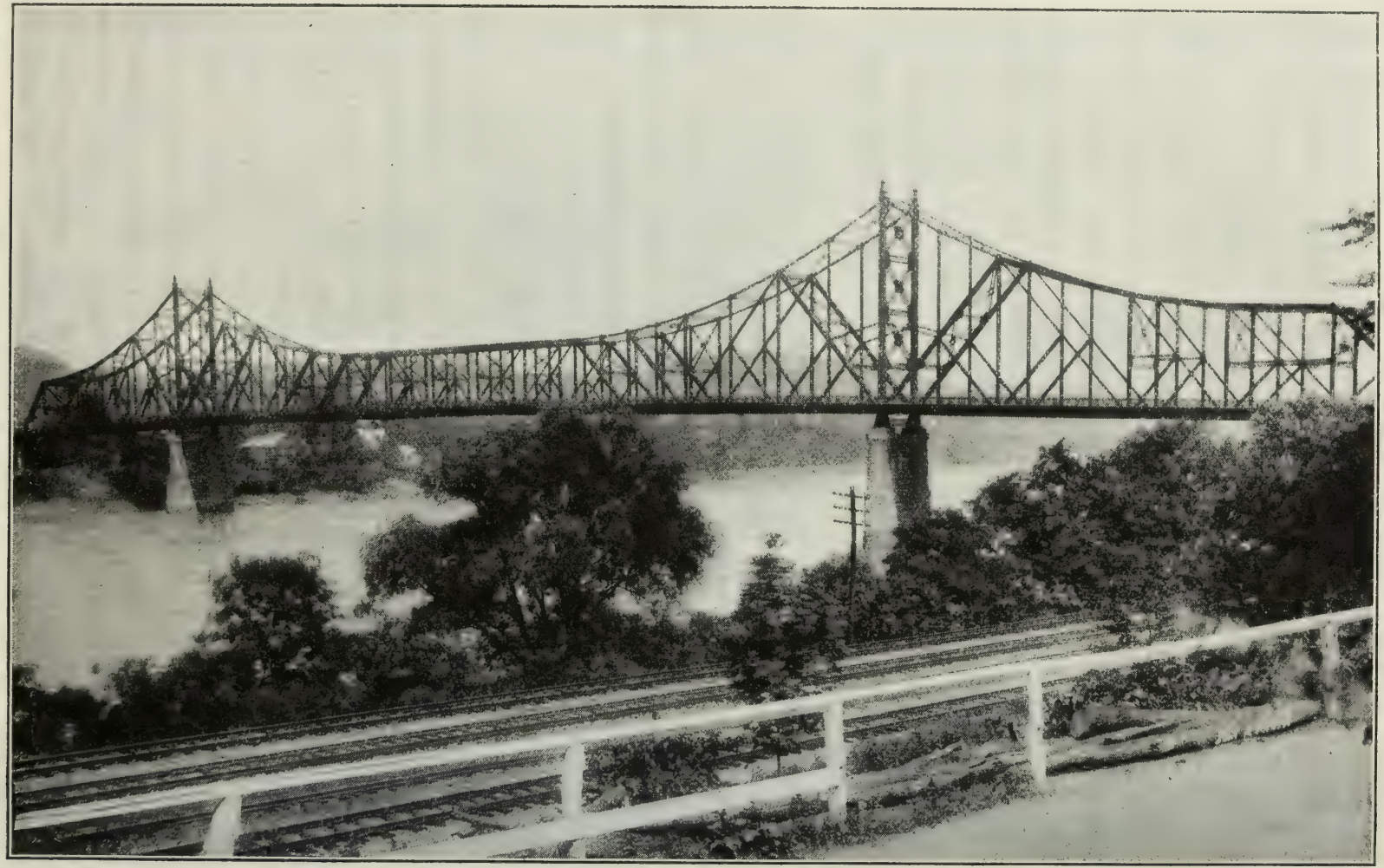


FIG. 13.—THE SEWICKLEY BRIDGE OVER THE OHIO RIVER. NEAR VIEW.

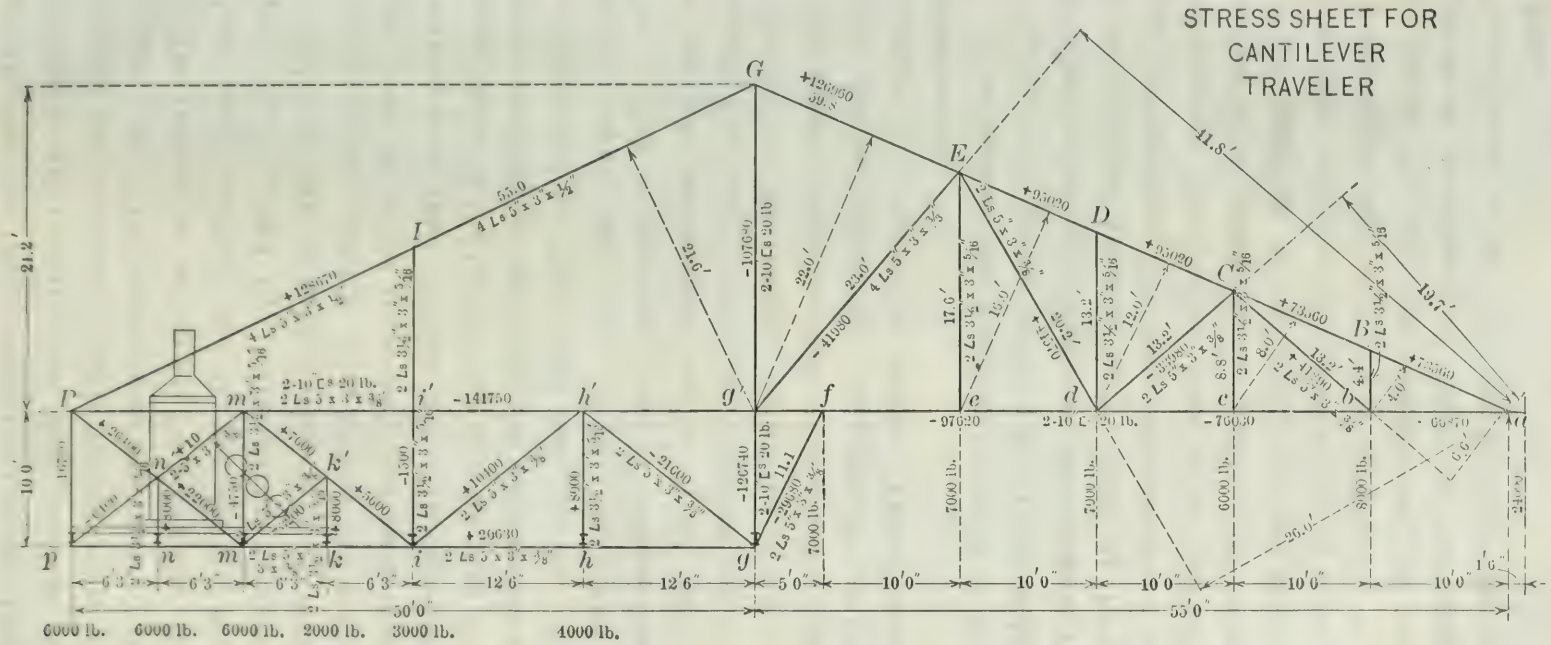
and the traveler track under the engines, because the sills were not sufficiently stiff to carry them when working. In place of the blocking, an extra pair of smaller wheels was afterward placed under the engines on each side. This improvement was made by the erection department.

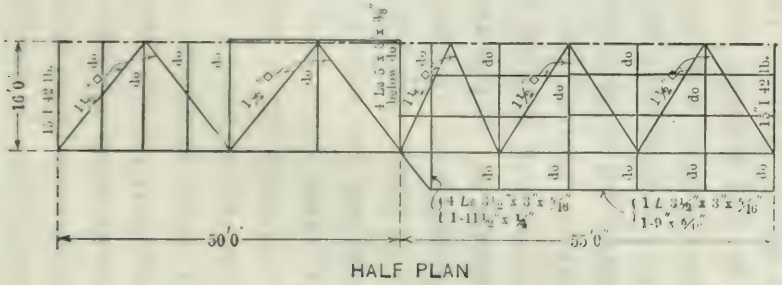
The traveler, as set up for the Sewickley Bridge, cleared the upper chord panel points, 8 and 12, but was not high enough to reach the pins, U_9, U_{10}, and U_{11}. The Bridge Company wished to avoid, if possible, the risk incident to rigging boom derricks at the elevation of the top of the traveler. The lifts at this point were quite heavy, the lower sections of the tower weighing more than 34 tons and the upper sections more than 31 tons each. The eye-bars for each panel of the upper chord at the tower weighed more than 13 tons. To meet this part of the problem, an extension was built on the rear end of the traveler, as illustrated on Plate XXI and Fig. 12. The loading girders were shifted toward the center, to clear the trusses, and carried, on their 7-ft. outlook or overhang, a pair of columns braced back to the girders, which, with an extension section on the rear wing braced back to the center wing and a batter post from the same, formed a gallows frame centered over the trusses. Extensions of the beams forming the top of this gallows frame provided attachments for light hoists to handle the pins, driving rams, etc. Supports for scaffold platforms were also arranged at a convenient height. The efficiency of this device met all expectations.

This traveler was designed for loads of 50 tons at each of the six wings at one time, in addition to its own weight and equipment. The saddles for the main hoist were also designed for loads of 50 tons. The loading girders are good for 23.3 tons at the center of a 37.5-ft. span, and for more than 50 tons on the outlook at a point 4 ft. from the center of support. The heaviest piece in the Sewickley Bridge was the lower section of the towers which weighed a little more than 34 tons.

In designing the loading girders and main hoist saddles, some consideration was given to the possibility of future electric equipment, and it is thought that there will be no difficulty in installing electric hoists on top of the main hoist saddles and, for the lighter hoists, at other points on the upper deck. With electric equipment, a controller cage, located so as to give the best field of view, would be installed in the wings.

Creeper Traveler.—Fig. 14 shows the stress, section, and loading



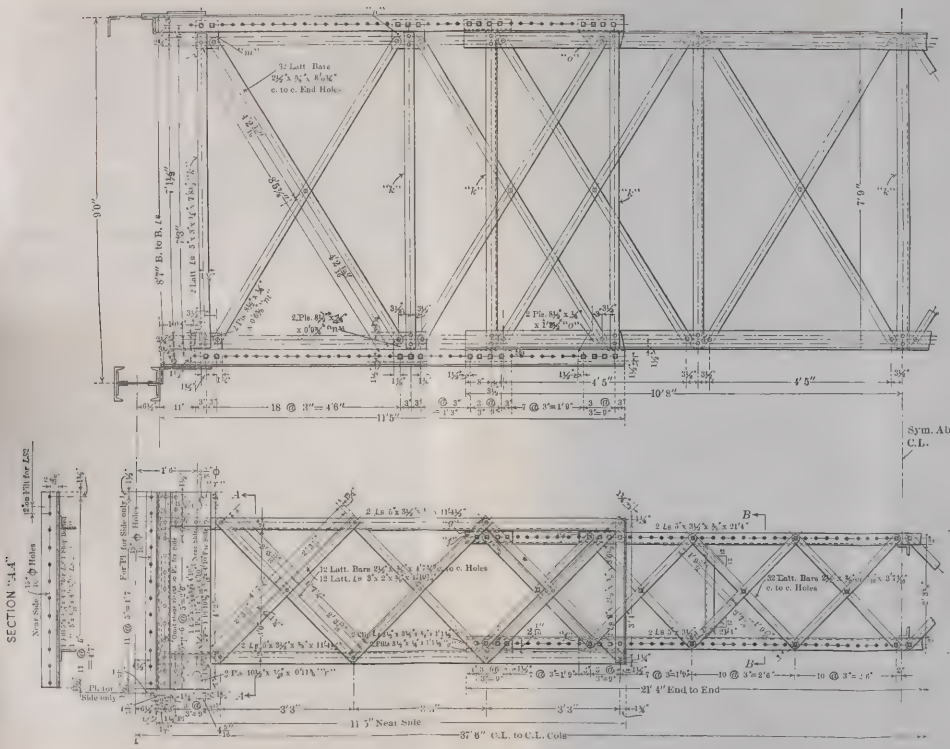


All Floor Girts 3½" x 3" x 5/16"

Dead Load, Cantilever Arm = 3 650 lb. per Panel per Truss
" " Tail " = 4 000 " " " " " 12½ ft. Panels.
Live Load, as indicated, including weight of Engine
Maximum Uplift = 25 860 lb.
Provide 50 000 lb, Minimum Anchorage for Each Truss

FIG. 14.

626A



DETAIL OF LONGITUDINAL STRUTS GANTRY TRAVELER

NOTE:-

Moment of Inertia	= 2308.0	Hor. Axis
Radius of Gyration	= 14.6	
Section Modulus	= 153.0	

Remove Lattice where clips interfere in adjusting to different Panel lengths, and replace lattice over clips if possible. If not practical to replace over clips, omit the panel of Lattice as, at clip, support is supplied by other section.

In adjusting to different panel lengths filler "g" under clip angles may have to be cut or else left projecting where lattice gussets come under clip angles.

Holts to be used where indicated thus ◘

All other parts to be riveted

diagrams of the creeper traveler. It was carried over the toggle bars on I-beam stringers supported at U_{16}, U_{17}, and U_{18}. From U_{18} it was supported on a track laid on the upper chord of the suspended span. This traveler was so simple that further description seems unnecessary. It met the expectations fully, and all the members of the erection force were pleased with it.

Credit for the results secured from the design and construction of this bridge is due to all the engineers who were connected with the work, both on the part of the County and of the Bridge Company, and the writer received material assistance from them in the preparation of this paper. G. Gudmundsson, Assoc. M. Am. Soc. C. E., Consulting Engineer, was associated with Mr. J. G. Chalfant, County Engineer, in the preparation of the contract plans. H. R. Blickle, M. Am. Soc. C. E., is Secretary and Chief Engineer of the Fort Pitt Bridge Works.

DISCUSSION

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—There is apparently nothing whatever to show why the cantilever type was selected for this simple bridge site. As far as one can see by the plans, there are no local physical conditions which call for it. It seems to the writer that three simple spans, approximating 450 ft. each, would cover the ground and furnish a stiffer, far better, and cheaper structure in every respect. He ventures to say, moreover, that the probable saving in first cost would have been fully 25%, if not more; consequently, he naturally cannot see the propriety of building such a structure at such a site.

It is natural to presume, therefore, that the taxpayers of Sewickley Borough and Moon Township, who paid for it, have substantial grounds for complaint.

Mr. Worthington.

Charles Worthington, M. Am. Soc. C. E. (by letter).—The writer would like to inquire what provision was made in this bridge to take care of the secondary stresses which develop in a cantilever truss of this type?

In the old Quebec Bridge, these secondary stresses were of such magnitude as to destroy the structure when the direct or axial stresses were about one-half the elastic limit of the material.

In the Beaver Bridge,* a very expensive roller bearing was provided under the main pier supports to reduce the secondary stresses.

In the Sewickley Bridge the members seem to have been proportioned for the axial stresses only. The writer has investigated a few of the members shown on Plate XIV, and finds that the sections agree pretty closely with those required by the stresses given on that plate and the unit stresses given in the body of the paper.

Consider, for example, the bottom chord member, L_{10}-L_{11}. Its length, l, is 900 in., and its radius of gyration, r, is 9.22 in. The unit stresses, as determined by the author's text, are then as follows:

$$\text{For live load, } 12\,000 - 40 \times \frac{900}{9.22} = 10\,040 \text{ lb.}$$

For dead load, double this, or 20 080 lb.

For live load, dead load, and wind load combined.

$$10\,040 \times \frac{20}{12} = 16\,750 \text{ lb.}$$

* "The Pittsburg and Lake Erie Cantilever Bridge over the Ohio River at Beaver, Pa.," by Albert R. Rayner, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 136

So that the required sectional area of the bottom chord member, L_{10}-L_{11}, is as follows: Mr. Worthington

Dead load.....	1 467 400	at 20 080 =	73.0	sq. in.
Live "	623 600	" 10 040 =	62.0	" "
			135.0	" "
Wind "	966 100			
Total	3 057 100	at 16 750 =	183.0	sq. in.

Taking the greater of these required areas, 183.0 sq. in., it will be seen that the actual area of 190.2 sq. in., is only 4% greater than that required, so that practically no excess of material has been added to take care of the secondary stress in this member, and the only provision for it lies in the unit stresses themselves. The writer does not think that 24 000 for dead load and 12 000 for live load, properly reduced for columns, are low figures for direct stress in steel of the character probably used in this structure.

The eccentric used at the L_0 point is in itself a source of secondary stress. If this eccentric were to be fixed in position so that the offset of centers of $\frac{1}{2}$ in. lies in a horizontal direction—a very possible condition—there would be developed at this point a bending moment of $240\,000 \times \frac{1}{2} = 120\,000$ in-lb., which, measured in terms of the area of the two 10 by $1\frac{1}{2}$-in. bars multiplied by the stress in the extreme fiber of the bar, is $120\,000 \times \frac{6}{10} = 72\,000$ lb.

The axial stress in these two bars is constant at 384 000 lb., so that this secondary stress of 72 000 lb. amounts to some 18.7% of the axial stress. This secondary stress may not develop while the bridge is new, on account of the lubrication applied to the eccentric, but it will certainly develop later. Taking the coefficient of friction between the pin and the bar at 0.4, the resisting moment to be overcome before the bar would rotate on the $9\frac{3}{4}$-in. pin is, $384\,000 \times 0.4 \times 4.87 = 750\,000$ in-lb., or about six times the amount necessary to develop the moment of 120 000 in-lb., due to eccentricity in application of the load to these bars.

Theodore A. Straub, M. Am. Soc. C. E. (by letter).—The results obtained from the methods used in the construction of the Sewickley Bridge were so satisfactory that a few words from the writer may be of interest. In all the departments of the Fort Pitt Bridge Company —the drafting-room, shop, and erection—the results were economical in the broadest sense of the word. These methods also aided materially in establishing that complete confidence and hearty co-operation which is so desirable and effective in the execution of work of this kind among all persons connected with it. Mr. Straub

Mr. Straub.

Duplication of structural steel members cannot often be controlled, but it is frequently possible to duplicate the parts which compose such members, even if there is a marked difference in their final make-up. This may or may not be the duty of the purchaser's engineer, but, if it is, the latter, by the stand he takes in settling questions of detailed shop drawings, can often make a seemingly inexpensive piece of work very costly to the fabricator, or *vice versa*. It will be noted that a special effort was made to secure such duplications, and the consequent economical results were due to the latitude allowed to the Bridge Company by the County Engineer.

Where possible, all field connections were reamed in their final relative positions in the shop. This resulted in securing good fits and finish, and obviated the necessity of the correction of mistakes in the field. The writer might say, in passing, that this is the general practice of the Fort Pitt Bridge Works on all work of any magnitude, and it has always been felt that it is an economical method of procedure. All field connections were made in the manner planned, without trouble and interferences, the final connections and adjustments being especially satisfactory.

The writer believes that the special angle lacing bar, as described by Mr. Buel, was used for the first time, as such, in an important bridge member. In sufficient quantities, its fabricating costs are not excessive, and the bars can be made in any blacksmith's shop which has a power hammer or press. This lattice bar is compact, efficient, and neat in appearance; it also has the additional advantage of readily shedding water.

Referring to Plate XIX, entitled "Profiles for Erection": These data were used freely, and were found to be especially helpful throughout the erection of the structure. They proved a ready check at all stages, and satisfied the engineers and erectors at all times that the work was progressing properly and safely.

The simple and inexpensive anchorage arrangement required the efforts of one man only in making the adjustment, thus giving extremely satisfactory results.

Both the wing and creeper travelers responded at all times to the duties planned and imposed on them in a very economical and efficient manner. It was also found that the adjustable features of the wing traveler, as well as the extra weight, did not affect the cost of its operation materially. The Bridge Company anticipates its use for future erection, and considers it a very good and efficient tool.

Complete harmony existed at all times among the engineers of the County and of the Bridge Company, and this was not the least of the pleasing results obtained by the methods used. All points in question were discussed freely and openly, and were settled promptly. Much credit is due to all in this regard and they deserve praise.

C. W. Hudson, M. Am. Soc. C. E.—It would be of interest to structural engineers to know how much it cost to make the anchor arms of this bridge self-supporting in case of a wash-out of the false-work. This expense, whatever it amounted to as a percentage of the whole cost, was in the nature of insurance against the loss of these arms, and against the loss due to the consequent delay in completing the structure in case of such wash-out. Mr. Hudson.

V. R. Covell, Assoc. M. Am. Soc. C. E. (by letter).—Referring to Mr. Le Conte's discussion of the type of structure used, the following conditions controlled: Mr. Covell.

The Act of Congress relative to bridges across the Ohio River, approved December 17th, 1872, as amended February 14th, 1883, requires that bridges above the mouth of the Big Sandy shall have at least one channel span with a clear waterway of 500 ft. between piers, measured at the low-water line, and with a clear height of 90 ft. above low water and 40 ft. above local highest water. The demands of the river shipping interests and the ruling of the War Department fixed the span at 750 ft., from center to center of piers. The local conditions were unfavorable for a suspension bridge, and the cantilever was adopted.

Henry H. Quimby, M. Am. Soc. C. E. (by letter).—Two of the features of the subject treated in this paper suggest some experience that may be of interest as a discussion. One is the construction of the floor, and the other is the deformation of the trusses in erection. Mr. Quimby.

The character of the floor construction of such a bridge is not the least important part of the design, because of the special value of lightness as well as of durability. The use of wood blocks is commendable, if the base be substantial. The paper does not state the size and character of the buckles of the floor plating. The thickness of the concrete paving base is said to average 3 in., which probably allows about 2 in. over the flat portions. If the plates are multiple-buckled, are the ridges or divisions between the buckles depended upon for cross-support, or is transverse stiffening provided?

Buckled floor-plates, as originally used, were in single buckles turned up, forming vaulted arches, and were supported on all four sides. Later, the custom arose of turning the buckles down and using long plates with several buckles in each, and without any cross-beams under the divisions. The strength of such a member is not easy to figure. It depends in part on the narrowness of the divisions, and in part on the thickness, and therefore the distributing value, of the concrete over them. The divisions must obtain support from the arch action of the inclined sides of the adjacent buckles. If a division be wide and have only a thin non-distributing paving over it, the corners or edges of the division will yield successively and alternately down and

Mr. Quimby. up under rolling loads. Only a slight movement of this character is required to crack and ultimately crumble cement concrete. When this stage is reached, the pavement is near to destruction.

Such a case occurred recently in the asphalt paving of a new deck on an old bridge where $\frac{5}{16}$-in. plates were used, supported on stringers, 5 ft. 6 in. apart in the clear, buckles, 5 ft. 3 in. by 4 ft. 3 in., and divisions varying from 1 to 12 in. in width because of varying panel lengths of the structure. Considerations of dead load and also of established street elevation demanded a thin floor, and so the paving base was used only to fill the depressions of the buckles, and the asphalt binder coat—1 in. thick—was laid directly on the flat parts of the plates. The wearing surface was 2 in. thick. The vehicle travel was very heavy, and, for several months during the reconstruction of the other half of the deck, it was concentrated on one shoulder in a narrow line. Within two or three months it was observed that the paving was breaking in several places, and it was found that these places were the wider, flat portions of the deck plates. In time the paving at two points was sufficiently pulverized to become dislodged and form holes, exposing the plate. Then the dropping of wheels off the edge of the surrounding paving pounded down one edge of each division thus exposed.

The remedy applied was the placing of chair supports under all divisions—a floor member of the old structure being suitably situated below each point—and then repaving the whole shoulder, the grade, at the same time, being raised $1\frac{1}{2}$ in., so as to have a little concrete over the entire surface of the floor-plates.

The lesson seems to be, that if multiple-buckled floor-plates are used without cross-support, the divisions should be as narrow as practicable, and they should be covered with a bed of concrete, say, as thick as the width of the division.

It would seem that the provision for making the field connections of certain members is not covered in the paper. $L - U_{11} - U_{12} - L_{12}$ form a quadrangle, made rigid by $M_{11} - U - L_{12}$, diagonally across it. The camber allowances in the lengths of these members must have distorted the quadrangle so much, while the members were without stress, that neither pilot point nor drift would avail to get the diagonal connected up, unless an easement were made somewhere. By what means was this connection made?

In the Red Rock cantilever bridge, where the main span is 660 ft., with a panel similar in character, the writer accomplished the connection of the panel by designing the lower chord splice below it so that it constituted a slip-joint. About 2 in. opening of this joint was required to permit the closing pin of the panel to be driven, and it remained open until the traveler advanced far enough in the erec-

tion of the succeeding panels to stretch and compress the truss members sufficiently to close the joint, and then it was riveted up. During the time that the joint was open the splice-plates prevented lateral displacement, and an adjustable bridle took care of possible tension from wind pressure. Mr. Quimby.

A. W. Buel, M. Am. Soc. C. E. (by letter).—The truss members of the anchor and cantilever arms and the suspended span of this bridge are pin-connected throughout, except where compression diagonals connect on forks forming common pin-plates at L_8, L_{10}, L_{12}, L_{18}, and M_{23}, and the connections of sub-verticals to the lower chords. The lower chords, from L_0 to L_{22}, and the top chords of the suspended span, are faced for square bearing under full load and spliced in the usual manner. Excepting the tower posts, $L_{10} - U_{10}$, which have 34-in. web plates, the maximum depth of these members is only 28 in. With these conditions, and reasonable care in detailing, no very large secondary stresses were found, and were not to be expected. Mr. Buel.

The 14-in. pin at L_{10} relieves the secondary stresses at that point in a manner fully as efficient as the segmental rocker bearing of the Beaver Bridge.

It is important to note that the weight and loads of the Beaver Bridge are about three times as great as in the case of the Sewickley Bridge; also, that the depths of main members of the Beaver Bridge are approximately twice as large as those of corresponding members of the Sewickley Bridge, and that similar deformations produce secondary stresses about in proportion to the depths. The same comparison holds substantially for the old Quebec Bridge.

In investigating the secondary stresses in the Sewickley Bridge, some important compensations developed, which made it unnecessary to increase the sections, except in a few instances. For example, take the secondary stress in the extreme fibers of lower chords, L_9, L_{10}, L_{11}, due to deflection of the trusses. With maximum live load on the bridge, the additional compression in the chords of the lee truss due to wind brings the stringers into action, relieving the chord. Another point that should be considered is the difference between the character of loading for railway and highway bridges, particularly as to the probabilities of maximum live load occurring simultaneously with maximum wind load.

The secondary stresses for all the points just mentioned, and for others, were investigated, and, considering the liberal provisions made for wind stresses, the comparatively small secondary stresses were sufficiently provided for, in the opinion of all the engineers connected with the work.

Regarding the eccentric flanged bushing in the pin-holes of the shoes at L_0, on the anchor piers, introduced to provide for adjust-

Mr. Buel.

ment, a very little study of the description and illustrations should be sufficient to show that no eccentricity at all is caused by this in the anchor bars. It is true that a maximum of $\frac{1}{2}$ in. eccentricity may exist in the bearing of the shoes on the masonry, but, as the base plates are 69 by 54 in., and as the maximum possible reaction produces less than half the permissible load per square inch on the masonry, the effect of this eccentricity is negligible.

It is not strictly true that the stress in the anchor bars is constant after adjustment is made. It will be relieved by the deformations of the masonry and of the shoe with its pin-holes and pin. Although this can hardly be computed with precision, an approximate estimate indicates that the variation would be about 20% from the maximum.

Answering Mr. Hudson's question as to the cost of making the anchor arms self-supporting:

The cost of this provision may be taken, approximately, as that of the following items:

Erection ties:

$U_2 - M_3$. Two 8 by $\frac{9}{16}$-in. eye-bars.
$U_4 - M_5$. One $8\frac{1}{2}$ by $\frac{5}{16}$-in. plate.
Four 3 by $\frac{5}{16}$-in. angles,
plus details, about 25 per cent.

Reinforcing, for erection stress:

$M_3 - L_4$. Two webs changed from 15 by $\frac{5}{16}$ to 15 by $\frac{1}{2}$ in.
Four angles changed from 3 by 3 by $\frac{5}{16}$ to 3 by 3 by $\frac{3}{8}$ in.
Pin-plates and areas back of and through pin-holes reinforced.
$M_5 - L_6$. Pin-plates and areas around pin-holes reinforced.

The temporary ties, $U_2 - M_3$ and $U_4 - M_5$, were only provided for one anchor arm, as they were removed from the north side before the south side was erected.

It will be seen that, in this case, the cost of making the anchor arms self-supporting was not great. Besides the sense of security that it gave to all concerned—by no means of negligible value—it left the Erection Department more latitude in arranging the sequence of operations, thus making for economy in the field.

Although it did not become necessary to swing the anchor arms on account of high water and ice, there were two or three occasions when the hazard would have been very great had it not been for this provision. Moreover, as a matter of convenience, they were swung and the falsework removed before the stresses were reversed.

Mr. J. G. Chalfant, County Engineer of Allegheny County, has furnished the following data on the floor construction, and his experience with a similar floor which has been in service several years:

"The buckle-plates are $\frac{3}{8}$ in. thick, from 18 to 34 ft. long, with several buckles in each, and are supported and riveted at the sides and ends, with the buckles turned down. The width of flat plate, between buckles, varies from 3 to 8 in., with a width of 11 in. in four plates. In 95% of the cases this width is not more than 6 in. The minimum thickness of the concrete is 3 in. Mr. Buel.

"The efficiency of this detail cannot be determined by calculation, and conclusions must be based on experience.

"Bridge No. 1, Allegheny River, at Oakmont, has substantially the same detail, and, after three years of service, shows no sign of failure."

The problem of making the connections in the field was given very careful study, the conclusions of which were entirely justified by the actual results.

Before adopting the lengths of truss members computed for camber, as described in the paper, a Williot diagram for cambered position was constructed, the vertical displacements being checked by computed deflections. From this a careful analysis was made of the erection conditions for each panel and joint, with particular reference to sub-panel members and to riveted field splices, such as those of the lower chord and main compression diagonals. The elevations of each bent of falsework and of the camber blocking were computed accurately, with allowances for settlement, etc., based on experience, as were also the positions of all lower-chord pins in relation to the traveler sills and rails. These elevations, with the clearances allowed for, were shown on erection plans. After this work was completed and checked, the engineers of the Fort Pitt Bridge Works were satisfied that no unusual difficulty would be experienced in the field.

The following quotations from the erection reports give the recorded results:

"As a rule, and speaking generally, in all riveted connections about 90% of the holes were filled with temporary bolts and drift-pins. Drift-pins were used in the field connections of members taking tension during erection, as bolts would not take up the clearance allowed in the holes, but all riveted members were connected practically without the use of drift-pins.

"All pins were driven home quickly and without the slightest distortion or displacement of metal about the holes. Special rams, weighing about 2 000 lb., were sent to the site, but were never used, as it was found that all pins could be driven easily with a 15-ft. section of 85-lb. rail."

Although it is recognized that the problem of providing for the field connections in bridges of such proportions as the Beaver, Blackwells Island, or Quebec is quite a different matter from that met in the Sewickley Bridge, it may be worth while to consider the causes of the rather unusual facility with which the latter went together in

Mr. Buel. the field, even when compared with structures in its own class. There is no doubt that the shop methods, as mentioned in the discussion by Mr. Straub, had considerable to do with it, and the form and type of trusses and details contributed to the results; but the methods used for introducing the camber and for checking its effect on field connections, together with the care taken to set the falsework and camber blocking to correct elevations, seem to be the most important items in accounting for the exceptionally satisfactory results attained. The accuracy of all these computations, as well as of the shop work, is attested by the remarkably close agreements between the computed and actual deflections, as shown on Plate XIX.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1256

STATE AND NATIONAL WATER LAWS, WITH DETAILED STATEMENT OF THE OREGON SYSTEM OF WATER TITLES.*

By John H. Lewis, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Clarence T. Johnston, L. J. Le Conte, George L. Dillman, W. E. Moore, Morris Bien, Horace W. Sheley, Morris Knowles, Kenneth C. Grant, E. G. Hopson, H. T. Cory, O. C. Merrill, William G. Davies, W. B. Freeman, and John H. Lewis.

In all irrigation, power, and other developments involving the diversion and use of water, much attention is given by the engineer to a detailed study of the available water supply. Whether or not legal title to the necessary water can be secured, is a matter usually left by the engineer for the consideration of the project's legal adviser. The lawyer, as a rule, is unfamiliar with the units for measuring water, knows nothing as to the effects on the stream of diversions or of return seepage, and can make but little use of elaborate hydrographs and water supply data prepared by the engineer. He confines his studies strictly to the legal phases of the problem as he understands them, assuming that the engineer will attend to what he considers the technical part of the work. The inevitable result is that the promoter, who usually understands only the financial end of the project and is relying on his engineer and legal adviser to do their part, suddenly awakens to the fact that he has expended a large amount of money on a project where the legal title to a part or all of the water supply is in some one else.

* This paper was first presented at the Annual Convention at Seattle, Wash., and again at the meeting of November 6th, 1912.

In the seven years' experience of the writer as State Engineer of Oregon, many disasters have been observed where the primary cause was clearly a failure on the part of the promoter, or the engineer, to appreciate fully at the outset the importance of investigating thoroughly those engineering questions which are semi-legal in nature. The fault usually lies with the engineer in making a water supply report which purports to be complete, but does not treat of the legal relation between the proposed diversion and all other diversions and rights along the entire stream. The engineer is much better fitted by training and experience to cover this disputed territory than the lawyer.

In many States, however, it is absolutely impossible for any one to secure definite information as to water rights or titles. The water laws of such States cannot be found by a study of the statutes alone, but must be sought for in a long series of decisions by the State Supreme Court. These decisions are based more on the logic of judges than the statutes of the Legislature, and are usually so conflicting that able lawyers differ in opinion as to the most fundamental points. In such States, it is the duty of the Engineering Profession to lead in the movement for the enactment of better water laws. It is impossible for the lawyer or statesman to frame proper laws without the co-operation and assistance of the engineer, for he alone knows the difficulties to be overcome in undertaking a new enterprise; and he, of all classes, should know what legislation is needed to protect all interests engaged in such an enterprise. The legal profession has great reverence for precedent, and, besides, there are many lawyers who feel that a reform in the water laws would affect business seriously, by reducing the amount of litigation. It is said on good authority that the cost of litigation over water rights in some of the Western States has exceeded the original cost of the works. The recent collapse of the irrigation bond market is evidence of the fact that capital has awakened to the importance of definite water laws, so that a reliable abstract of water title can be prepared prior to construction. Therefore, looking at this matter only from the selfish standpoint, it is the duty of the engineer to lead in the reform of State and National water laws, so that the confidence of capital may be speedily restored and that development may proceed. The best lawyers in every community will always be found on the side of

reform, for they realize that their prosperity depends on the general prosperity of their community and State.

The wonderful progress made during the past decade in the reclamation of arid lands, through the agency of the Carey Act and the United States Reclamation Act, has served to educate the public as to the value of water and the importance of some interstate or National laws relating to diversions from interstate streams, or their tributaries. Formerly, hundreds of small diversions were necessary before any appreciable effects on a large interstate stream were noticeable in the lower State, but now, in some cases, the entire flow of a great river may be stored for several years in a reservoir, and the flow in the adjoining State is affected so suddenly as to be a shock and a menace to its citizens. Such enormous developments were not contemplated at the time of the framing of our State and National constitutions, and some new legislation must be framed and enacted to cover these contingencies, even if it necessitates the stretching or amending of these constitutions.

Some interstate legislation is absolutely essential if the highest development of our streams is to be accomplished. Each river, from its head-waters to its mouth, should be treated as a unit, regardless of State lines. For instance, it should be possible to store water in Colorado for use in Southern California, the stored water being protected by Federal or interstate law as it passes through three intervening States. Again, the construction of a power-plant, to utilize the entire low-water flow of a stream in one State, may interfere seriously with subsequent diversions for irrigation in the adjoining State immediately above such plant. A number of cases of litigation between States over a proper diversion of interstate streams have already been carried to the Supreme Court of the United States, and some definite action should be taken by Congress before these decisions become hopelessly confused, as is the case in most States where no statutory laws have been available for the guidance of the Courts. The National engineering societies should lead in the discussion of this important subject, and assist in the framing of necessary interstate water legislation.

Just as we have done away with county control of waters, for State control in Oregon, so must we supplement State control by some system of interstate or Federal control for interstate streams,

and the same system which has been found satisfactory for the State should be extended to interstate waters.

The water laws of Oregon, though not specifically referred to, have been endorsed as a model for other States by resolutions of the National Irrigation Congress. It required a four-year campaign, supplemented by the powerful club, the power of direct legislation through the initiative, now in the hands of the people of Oregon, to force this law through the legislature. It has now been in successful operation for three years, and the results accomplished have been far in advance of what was hoped for, even by the most sanguine of its supporters.

For the foregoing reasons it is believed that a brief statement of the fundamental principles underlying the Oregon system of water titles, together with the results thus far accomplished, will be of interest to the Engineering Profession generally, and may assist in the solution of some of the conservation and water-power problems which are now calling for the highest degree of statesmanship on the part of our representatives in Congress.

The value of this paper will be greatly extended if it receives careful consideration and full discussion by engineers in the Eastern, Southern, and Central States, as they look at these questions from an entirely different view-point. The general impression seems to prevail that a law suitable for the Western arid States would not be adapted to conditions in the Eastern humid States. It is high time that this notion be dispelled; for the diversion or storage of 100 sec-ft. of water from a stream for irrigation or domestic use in Oregon would have the same effect on lower appropriators as a like diversion for an eastern stream for any similar uses to which water may be put in such section. Until recently, the common law doctrine of riparian rights, in all its rigors, applied to conditions in the western or humid portion of Oregon, while the doctrine of appropriation for beneficial use prevailed in the eastern or arid portion of the State. The same law has now been made applicable to all parts of the State.

The writer will now endeavor to show that this same system of water titles, which is essential to development in the arid States, is equally applicable to Eastern and Southern States, so that this discussion may be of interest and perhaps of value to engineers in all parts of the country.

"The United States inherited from Europe two great systems of jurisprudence, one the Common Law, and the other the Civil Law. The eastern part of the territory of the United States began with the Common Law. The territory west of the Mississippi inherited the Civil Law, through the prior sovereignty of either Spain or France, or both. The Common Law has invaded most of the area of the Western States, formerly subject to the Civil Law, yet there are many great questions of both title and right still dependent there for settlement upon the rules of the Civil Law. Among the great questions thus dependent is the law of waters.

"The basis of the Civil Law which comes to any State of the American Union, is the law of imperial Rome. It will surprise any one who reads the translation of the Roman water law to see how perfectly and fully they understood the whole question. They had worked out every problem, and we may safely say that they understood the subject as well as we do to-day, and no one can say that the old Romans did not bring to their law of water a common sense and equity not exceeded by our courts to-day."*

The modern water laws adopted by a few of the Western States, including Oregon, are based on the Roman Law. The basis is priority of appropriation and beneficial use. The Oregon territory, however, was acquired by discovery, and therefore this State started off with the same system of jurisprudence—the Common Law—as the eastern part of the United States. The early decisions of our Courts were in harmony with decisions in the Eastern States, yet these decisions have been almost completely reversed in a State where occurs, perhaps, the heaviest rainfall (in Tillamook County, 139 in.) of any point in the United States. The writer, therefore, cannot see any reason why a law similar to that of Oregon cannot be adopted and upheld in any other State of the Union where it is necessary to divert water.

Our system of land titles is well understood in all States. We know the value of an abstract of title. Why should we not have a definite system of water titles in all States, and why not insist on just as definite showing of water title before the purchase of a water project, or before the investment of money in a new enterprise? Entire streams are now being diverted or stored in the Eastern States for municipal supplies, for supplying large canals, for the development of power, etc. In many cases, water is carried from

* Ware, on Roman Water Law.

one stream into another and from one State into another. Soon the State and Nation must join in the storage of water for the control of floods and to aid navigation. This water, in passing down the stream, will benefit many private power projects, and these should be compelled to contribute to the cost in proportion to the benefits received. It is the writer's belief that definite State and National laws relating to water would be of as great benefit to the Eastern as to the Western States.

Riparian Rights *versus* Appropriations.

Before going farther into this subject, it might be well to explain briefly the distinction between the common law doctrine of riparian rights, and the civil law doctrine of appropriation and use.

The common law doctrine of riparian rights originated in England, where it was not necessary to divert and use great rivers for the domestic supply of cities, where no extensive diversions were necessary for the supply of canals, or for water-power projects, as we know them to-day, and where irrigation was not necessary. It was more a question of drainage than of irrigation.

"Riparian rights" are such as follow or are connected with the ownership of the banks of streams or rivers.

"Every proprietor of land on the bank of a river has an equal right to the use of the water which flows in the stream adjacent to his lands, as it was wont to flow, without diminution, pollution or alteration, unless his right has been limited by grant, license or prescription. No proprietor along the bank of a stream, however, has a right to use the water to the prejudice of other proprietors above or below him, unless his right has been enlarged by grant, license or prescription. He has no property in the water itself, but a simple usufruct while it passes along. * * * Though he may use the water while it runs over his land, he cannot unreasonably detain it, nor give it another direction, and he must return it to its ordinary channel when it leaves his estate. These rights are inseparably annexed to the soil, and pass with it, not as an easement or appurtenance, but as a part and parcel of the land. This property right can be regarded only as a corporeal hereditament, belonging to and incident to the soil, the same as though it were stones or grass. The right does not depend on appropriation, and is not suspended or lost by nonuser, but may be lost by grant or prescription, or long adverse use; or in the semi-arid states, by condemnation."*

* Mills' Irrigation Manual, p. 18.

The doctrine of appropriation for beneficial use, as far as the United States is concerned, originated in California and other Western States, out of the necessities of the miners and early settlers. It is based on the familiar maxim that "he who is prior in time is prior in right." Water is treated as a wild thing which becomes the property of him who captures it first. It can only be retained by him so long as he puts it to beneficial use. If the supply is not sufficient for all, the ditch of the last appropriator must be closed to protect the prior appropriator. This system of titles is simple, easy of enforcement, and is the only system which is adapted to the needs of the people in this age of big developments.

These two systems are in direct conflict, and cannot both be applied on the same stream at the same time, for streams cannot flow undiminished to the ocean, and at the same time be diverted for beneficial use.

"From the history of this subject, dating * * * from the earliest period of Egyptian history, down to the period when extensive appropriations were made for mining and agriculture in the arid and semi-arid West, we are unable to find any controversy between those claiming as riparian proprietors and those engaged in diverting and conducting the water to non-riparian lands. It seems to have been an accepted fact that the water was the property of the public, and when the necessities of the people required that it should be conducted from the stream and applied to the soil for the production of crops, the right to do so was unquestioned."*

Early Laws.

Oregon was admitted as a State on February 14th, 1859. Its constitution contains no provisions relating to water. Many decisions had been rendered by our Supreme Court before the adoption by the legislature of the first water laws, in 1891. These laws were in harmony with the customs of the miners, and provided for the posting of a notice on the bank of the stream, setting forth the extent and purpose of the claims, and the recording of a copy of such notice with the county clerk. Work was to begin within six months, and no time limit was fixed for completion. There was no supervision by the county or State. As a result, each applicant usually claimed many times the water needed, and usually all the water in the stream at

* Mills' Irrigation Manual, p. 6.

that point. The records, therefore, were of no public value, and in one case a trip of 1 000 miles would have had to be made to consult all the records relating to a single stream. Litigation was constantly in progress on some streams. The decrees settled only the rights of the parties interested, and were not final because they did not involve all parties on the stream, including the State, which is vitally interested in the unappropriated waters, as trustee for future users. This litigation impoverished the water users, discouraged settlement and investments, and promoted community quarrels which threatened the peace and safety of the citizens. The term "water right" became in practice synonymous with litigation. To overcome these difficulties, a complete system of State control was provided in February, 1909, and is based on the police power of the State to preserve the peace and safety of its citizens.

The New Law.

All water within the State of Oregon was declared by law to be the property of the public. This declaration, it is believed, merely states the law as it exists to-day in every State of the Union. It is the purpose of this new law to regulate the use of this property in the interest of the public, without injury to prior vested rights. This regulation is possible under the police power of the State.

A board, of which the State Engineer is chairman, was created to have charge of this property.

It is the duty of this board:

A.—To determine and record all rights to the use of this public property which had become vested prior to the adoption of the law;

B.—To grant rights, for beneficial use in the State's unappropriated waters, to those who make proper application therefor;

C.—To protect recorded rights to the use of public waters by regulating diversions from streams.

The law and its operation will be discussed under these three heads, taking them up in order.

Determination of Old Rights.

A complete and reliable record of all old rights is the foundation for their protection as well as the basis for the State's administrative

system. Such a record is essential, in order that the public may know the extent of its unappropriated waters. As streams do not follow county lines, or begin and end in particular judicial districts, it was found absolutely necessary to create a special tribunal to ascertain and record these old rights. It is called the Board of Control, and is composed of the State Engineer, and the Superintendents of each of the two sections into which the State is divided.

The Board of Control is an administrative body having only subordinate judicial powers. While it has much the same powers as a Court, and follows, where practicable, the usual legal procedure, yet it is free to depart from such procedure where necessary to facilitate action. It operates in much the same manner as the Railway Commission, or the Board of Health. It knows what information is necessary for a proper determination of early rights. It goes after this information, and does not accumulate thousands of pages of useless and conflicting evidence. Its success has been due largely to its freedom in this respect. Any water user can submit his claim to the Board, if he so desires, without the necessity of employing an attorney. The Board assists him by preparing blank forms* and by making surveys and maps, so as to minimize his expense. In case of contests, the Board pays for the taking of additional evidence.

The determinations are not only inexpensive, but speedy and effective, as shown by the record. The members of this Board are employed by the people of the State for this particular work. It is to their interest, as well as that of the water user, that these determinations be made speedily, and that the determinations are effective.

The first step in the adjudication of water rights on any stream is to file a petition with the Board, signed by one or more water users, requesting that a determination be made. If conditions justify such action, and funds permit, the State Engineer is directed to make a survey as a basis for such determination. Maps are prepared showing each ditch, the location and extent of land irrigated, and such other information as may be necessary in defining the right.

Next, a form containing questions is sent by the Superin-

* A full set of the forms and instructions used in Oregon was furnished by the Author, but as they are too voluminous to be published here, they have been filed in the Library of the Society, where they may be examined by those especially interested.

tendent to each person or company claiming a water right from such stream. Each claimant is required, under penalty of forfeiture, to fill in the information necessary for a proper determination of his right. The Superintendent, at the time and place fixed in the notice, proceeds to collect these statements, which are certified under oath before him. The maps prepared by the State Engineer are at such time submitted to inspection by the water users, who must accept or reject them. If not accepted, maps must be prepared and submitted by the claimant showing the information as he thinks it should be. Certain fees are payable at this time.

After all claims have been filed on these forms, a second notice is sent by registered mail to each water user, fixing a time and place when these claims will be submitted to the inspection of all interested parties. Within 5 days after the closing of this period of inspection, any claimant may contest the claim of another, each being required to pay $5 for each day required in the taking of testimony in such contest. The deposit of the winning party is returned to him. In this way, the determination of early rights is practically left to the water users themselves, and trivial contests are discouraged. The new comer will contest vigorously any exaggerated or untruthful statement by any prior appropriator. The more these early excessive appropriations are cut down, the more valuable is his own right, and the more likelihood of his securing water. The early appropriator, knowing that his claim will be submitted to the inspection of his neighbors having subsequent rights, is usually modest in his claim.

The original evidence, with that taken in contest cases, is then submitted to the Board of Control, and, as soon as practicable, an order determining the relative rights of all parties is entered. This order becomes effective immediately, and can be enforced by the appointment of a water master.

The order, together with all testimony collected by the Board, is filed later with the Circuit Court of the proper county for confirmation. If no exceptions or appeals from the Board's determination are filed with the Court, an order is entered confirming the same. Otherwise, the decree remains in force until the exceptions or appeals are tried out, and a modified decree is entered. Appeals from the decree of the Circuit to the Supreme Court are provided for, if

taken within six months from the entry of the decree in the Lower Court.

On confirmation by the Court, and after the period for appeal has expired, a water right certificate is issued by the Board of Control to each owner of a right.

There is considerable demand for making the decisions, final with the Board, subject to appeal to the Courts, in the usual manner. Such procedure would greatly simplify matters, and hasten materially the issuance of certificates. It would be clearly constitutional, in the writer's opinion, as indicated by decisions in Wyoming and Nebraska.

Water Right Certificate.

The water right certificate is record evidence of the holder's right to water. It is to his water title what a patent from the United States is to his land title. Thereafter his title to water for irrigation passes with the land on which it is used. An abstract of land transfers will then serve to show chain of title to both land and water.

This certificate, which is issued to the claimant by the Board after final determination of his right, embodies all the fundamental principles of a water right. They are few and simple, and, if kept clearly in mind, will give a good understanding of the water laws of Oregon as well as of a number of the Western States.

These fundamental principles are as follows:

Beneficial Use: { Priority, Purpose, Period, Place, and Quantity.

Beneficial use is the great underlying principle. Because of this principle, there are no water monopolies in Oregon. The barter and sale of water, apart from the use, and the holding of water without use, for speculative purposes, are unknown. Our water resources, greater perhaps than those of any other State in the Union, are still largely the property of the public, and are offered practically without price to him who will put them to use.

Priority.—The priority is expressed by a date. It is the most important element in considering the value of a water right. It is the date when the present owner or his predecessor first diverted

water or took the first legal steps to initiate his right. A ditch may have several priorities, if the original appropriation or intent has been enlarged at a later date.

Purpose.—The purpose, whether for irrigation, power, mining, manufacturing, or domestic use, is an important element of the title. A right acquired for one purpose should not be transferred to another without loss of priority. A change from power, which does not consume water, to that of irrigation, which consumes a large part of the water, will affect lower rights seriously. Such changes are against public policy. The user has no property right in the water, only a right for a particular use.

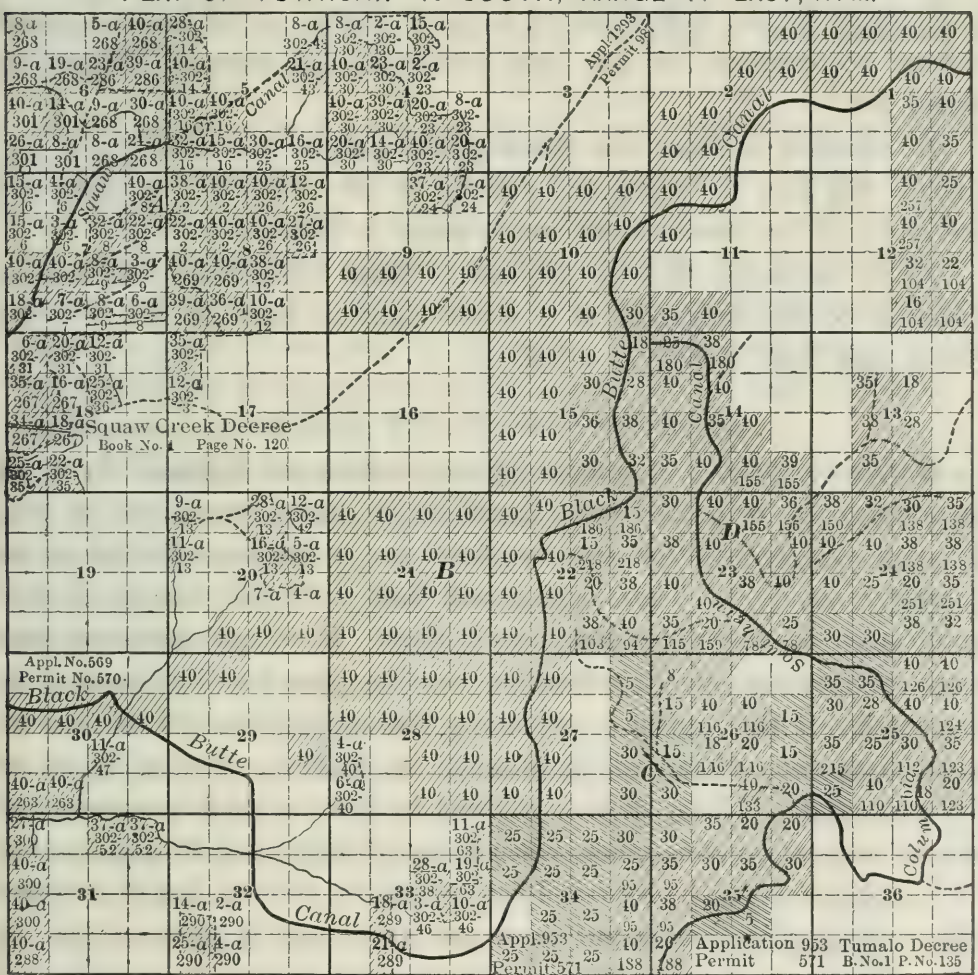
Period.—The period of use is important. A right for mining during the winter months cannot be extended to include summer use, as this would conflict with perhaps prior rights for irrigation. A right for spring irrigation of grain, and long used for such purpose, cannot be enlarged to include fall irrigation of alfalfa, without injury to others. The increased use at a different period would damage vested rights below.

Place.—The place of use is fundamental to a proper definition of the right. Water cannot be branded or fenced like cattle to show private ownership. The nearest approach to such system is to tie it to the place of use. Therefore all water should be made appurtenant to the place of use, otherwise there can be no stability of water titles, and the public records at Salem will be of little use. (See Fig. 1.)

No transfers should be permitted without loss of priority, except in the most extreme cases, and then only after proceedings and record before the Board of Control. The return water from prior appropriations is counted upon by subsequent appropriators as a basis for their investment. Even a relatively small displacement will affect the return seepage, as the laws of underground water are uncertain. There could be no stability of water titles if the law permitted indiscriminate transfers in the place of use. If the use terminates, the right should terminate. Even if a transfer is made after notice to all parties, it is unfair to the lower appropriator, as anticipated injury can seldom be proven. The public gains nothing, as new appropriators are always eager to put such water to new uses under a subsequent priority.

To permit a change in character of use, or change in the place of use, therefore, would serve to make water titles unstable, and open the way to the barter and sale of water, which is in direct conflict with the doctrine of beneficial use, on which the entire system rests. Unless water titles can be made definite and certain, capital cannot be expected to invest in new enterprises.

PLAT OF TOWNSHIP 15 SOUTH, RANGE 11 EAST, W. M.



A—Decreed rights from Squaw Creek for which water right certificates have been issued.
B—To be watered under permit from Blue Lake and Metolius River.
C—To be watered under permit from Crater Creek and storage in Tumalo Reservoir.
D—Incomplete rights under Tumalo Creek adjudication; project not yet completed, including storage in Tumalo Reservoir.

FIG. 1.

Quantity.—The maximum quantity of water granted in the certificate is an important qualification to the general limitation of beneficial use. This is expressed in cubic feet per second, which is the standard unit for rate of flow. Water for storage purposes is ex-

pressed in acre-feet, which is the unit of volume. If a greater quantity than allowed is desired, this can be delivered by the water master under a rotation system, that is, where two or more combine, each using the total allotment, part of the time. If the water is being wasted, the water master is authorized to close the gate to such extent, permitting such water to flow down to others.

The point of diversion, or even the location of the ditch, is almost immaterial in the definition of a water right, as its location must change to conform to the new conditions of channel caused by floods, etc.

In addition to these five fundamental points, the water right certificate will give the name and address of the owner of the right and the name of the stream from which water is diverted.

Much confusion will be avoided if these fundamental principles, essential to the definition of a water right in some of the Western States, are kept clearly in mind.

The Granting of New Rights.

Rights to the use of unappropriated waters are initiated by filing an application with the State Engineer, on forms prescribed by him. Priority of appropriation and beneficial use is the basis for all rights hereafter acquired. These rights will date from the receipt of such application in the State Engineer's office. It is not necessary that maps accompany such application, or that complete information be presented. If the project is extensive, the application will be held for a sufficient time within which to make the surveys, prior to its return for correction. Thirty days is allowed by law for such purposes, and special request must be made, when filing, if additional time is wanted. The application can be returned for correction without loss of priority. When the application is put in proper form and is approved by the State Engineer, it becomes a permit, and the applicant is authorized to proceed with construction.

No right to the use of any public waters can be acquired without making such application and securing a permit. This is also true for the extension or enlargement of existing rights. Riparian owners must comply with the law the same as others. The owner of a spring tributary to a running stream must also make application. A reliable record of all new rights or extensions is absolutely essential for proper State control. It is the basis for new investments, as well as for the

protection of prior rights. To insure the reliability of this record, the following penalties have been provided:

"It shall be a misdemeanor to use, store, or divert any water until after the issuance of a permit to appropriate such water.

"Any person who shall willfully divert or use water to the detriment of others, without compliance with law, shall be deemed guilty of a misdemeanor.

"The possession or use of water, except when a right of use is acquired in accordance with law, shall be *prima facie* evidence of the guilt of the person using it."

No application will be approved for more water than can be applied to beneficial use, and the State Engineer is authorized to limit an application to a less quantity of water than that applied for, if there exists substantial reason therefor. During the past three years all applications for the use of water for irrigation purposes have been limited to 1 cu. ft. per sec. for 80 acres. An application may be denied where the proposed use conflicts with determined rights, or is a menace to the safety or the welfare of the public.

Work must be commenced within one year from the date of approval of the application, and be completed within a reasonable time, as fixed in the permit, not to exceed five years. Additional time after completion is allowed within which the water must be completely applied to beneficial use. Annual proof must be filed with the Board of Control to show progress of work under the permit. If the water is applied to beneficial use within the time allowed, final proof is taken of such fact by the Division Superintendent, and a survey is made in order to ascertain the extent of the use. Based on such proof and information, the Board of Control will grant to the applicant a water right certificate.

This certificate will be of the same form as that issued to early appropriators on final determination of their rights. The form of this certificate has been fully described above.

The fees paid to the State for the privilege of appropriating its public waters are small, but in the aggregate more than equal the appropriation for the office and department of the State Engineer.

Protection of Rights.

The primary object of making water rights definite and certain, through adjudications by the Board of Control, and through permits

issued by the State Engineer, as outlined herein, is to furnish a proper basis for the protection of such rights by the State.

The accomplishment of this object furnishes at the same time the only safe and reliable basis for new investments. Only the more expensive projects remain for development. For this reason no State which fails to provide such a system of water records, and public supervision of stream diversions, can expect to secure the highest development of its water resources. Private capital will not invest without such records and information, and public funds should not be invested, for obvious reasons.

To facilitate the distribution of water, the State has been divided into two parts, over which is placed a superintendent. These divisions are cut up into districts, as rapidly as the various rights are determined and the need thereof arises. On the petition of one or more water users of such district, a water master may be appointed by the Board, on the recommendation of the superintendent.

It is the duty of such water master to regulate the head-gates of ditches so as to insure the delivery of water in accordance with established rights, and to prevent waste. He has authority to make arrests, and to compel the installation of the necessary head-gates and measuring devices. When the water supply falls so low as to be insufficient for all, the ditch of the latest appropriator, having the poorest right, will be closed, so that prior rights may be protected. As the water falls, other head-gates will be closed, and these in turn will be opened in case of a rise in the river.

If water is stored at the head of a stream, and the natural stream channel is used in conveying such water to the place of use, it is the duty of the water master to set all head-gates so as to admit only the quantity to which each ditch is entitled, in order that the reservoir owner may recover at the place of use the water he turns into the stream, less the quantity lost by seepage and evaporation in transit.

Protection of the Public Interest.

Public interest demands that water be put to the highest use. In case of conflict between power and irrigation, the application for the former use can be referred to the Board of Control, on the ground that it is a menace to the safety or the welfare of the public, and the Board has power to direct its refusal, if public interest demands.

Likewise, an application for either irrigation or power can be denied if it is in conflict with the higher use for domestic supplies.

Oregon's Supreme Court has upheld the Board of Control on this interpretation of the law in the case of Cookingham *vs.* Lewis.* Recently, the Board directed the refusal of a prior water-power application which menaced the water supply of the City of Nehalem.

Water-power permits have not been issued on streams which are more valuable for irrigation purposes than for power, and where an abundance of power can be generated on the tributaries of such stream above the points where diversions for irrigation are possible. A comprehensive plan for the development of the State's water resources is essential, if waste is to be avoided and the public good promoted. To permit the construction of one or two low dams for the utilization of all the waters of the stream at a point which is the only practical place of diversion for irrigation will greatly retard development. The public, in permitting such power development, is selling its birthright for a mess of pottage. With a 15-ft. dam, 1 000 sec-ft. will generate 1 700 h.p., but, if held available for irrigation, would reclaim more than 300 000 acres of land with storage. The power grant, as far as the public is concerned, therefore, is equivalent to a free grant of 300 000 acres of land. It is true that the power grant may later be condemned, but why grant the franchise and delay the development only to buy the same later at an enormous franchise valuation in addition to its physical value, when the same power could easily be generated at a little greater expense on the tributaries of this stream, where no conflict with irrigation would ever occur. From the standpoint of taxable property, the relative merits of those two conflicting uses would be as thirty thousand is to thirty million.

Typical Water Record, Under New System.—Fig. 1 is a plat showing the clearness with which the character and extent of water rights can be defined for each 40-acre tract when the water is made appurtenant to the place of use. A similar plat for each township in the State is kept in a loose-leaf book, and is indexed for quick reference. On these plats all information relating to water is compiled as it accumulates.

If a prospective settler should write the State Engineer asking if the S. E. ¼ N. W. ¼, Sec. 18, T. 15 S., R. 11 E., had a water right, a

* 114 Pacific, p. 88.

reply could be dictated in a few seconds giving definite information that such tract had a vested right to water for 16 acres from Squaw Creek under Water Right Certificate No. 267. Such definite records make present rights more valuable, encourage settlement, and new development. The issuance of overlapping permits can be avoided. The making of water appurtenant to the land irrigated is the basis for this system.

The plat, Fig. 1, shows three different classes of water rights, as explained below the cut. It is peculiar in that no stream crosses its border, yet the lands will receive water from four different streams.

Lands not shaded must remain forever without water. Lands *B*, *C*, and *D* will not receive water unless the contemplated works are constructed within the time allowed. Purchasers buy with notice of this fact. A published record, by stream systems corresponding to this township plat record, will eventually be issued for gratuitous distribution. It will give the Priority, Purpose, Period, Place, and Quantity for each recorded right, with the name and address of the owner. Such a record of vested rights, together with a published record giving the total available supply for corresponding streams, is essential as a basis for the development of our water resources.

Results Accomplished.

The results accomplished during the first three years of public control in Oregon should demonstrate beyond question the value of the new system of water titles. No serious defects in the law have been discovered. It became effective on February 24th, 1909. Much of the first year was devoted to the preparation of forms and records, and outlining the system. Without a definite and comprehensive system, there could have been no hope of bringing order out of the former chaotic condition of water titles.

Old Rights Determined.

In three years, ninety-one petitions have been filed with the Board of Control, asking for the determination of old rights, so that the benefits of State protection could be had.

Surveys looking to this end have been undertaken on only twenty-three different stream systems. The Board is at present several years behind in its work. The present organization could handle a far greater volume of work if additional funds were available for employ-

ing clerical and technical help. The system is simple, but the filing, indexing, and tabulating of rights, and the correspondence with the numerous claimants, involve an enormous quantity of clerical work which requires office assistants who have had several years' experience in such work.

Thus far, complete adjudications have been made by the Board on fifteen different streams, involving 965 separate rights. These determinations affect title to water for the irrigation or 89 084 acres of land, at a total cost to the claimants of $9 119.

TABLE 1.—Water Right Adjudications—Confirmed.

Stream.	County.	No. of rights involved.	No. of rights decreed.	Irrigation Rights.		Fees paid.	Average cost per right.
				Vested rights, in acres.	Incomplete rights, in acres.		
Willow Creek	Gilliam and Morrow	204	163	5 936	355	$2 056	$12.60
Mill Creek	Union	115	84	2 035	.....	659	7.85
Squaw Creek	Crook	110	80	7 072	9 408	1 085	13.56
Tumalo Creek	Crook	149	109	3 058	25 120	854	7.84
Paulina Creek	Crook	23	17	530	174	193	11.33
Cochman Creek	Grant	8	5	35	.....	25	5.03
East Branch Mud Creek	Umatilla	19	17	532	.....	156	9.15
South Branch Mud Creek	Umatilla	19	15	445	.....	114	7.60
Goodman Spring Branch	Umatilla	11	6	202	.....	59	9.79
Totals	.....	658	496	19 845	35 057	$5 201	$10.50

Water Right Adjudications—Complete but Unconfirmed.

Stream.	County.	No. of rights involved.	Irrigation Rights.		Fees paid.	Average cost per right.
			Vested rights, in acres.	Inchoate rights, in acres.		
North Powder River	Baker	134	20 496	499	$2 244	$16.74
Butter Creek	Umatilla and Morrow	62	7 712	.....	1 062	17.13
Cottonwood Creek	Malhuer	4	173	.....	58	14.61
Sucker Creek	Josephine	87	2 162	2 391	428	4.92
Althouse Creek	Josephine	20	749	.....	126	6.31
Totals	.....	307	31 292	2 890	$3 918	$12.75

Nine of these determinations have been confirmed by the Circuit Courts in the respective counties, and 496 certificates have been issued. The average cost in fees to these certificate holders has been $10.50, and

the total area of irrigated land included in these final records amounts to 54 802 acres.

These nine adjudications have been completed without a single appeal to the Supreme Court. The streams involved are among the most complicated in the State. Heretofore, practically every water case was carried to the highest tribunal, and final decisions were delayed from five to ten years.

Surveys have been completed by the State Engineer and maps have been prepared showing all water right locations on twenty-three different streams, including the fourteen mentioned in Table 1. These surveys include 265 055 acres of irrigated land, or approximately 40% of the total irrigated area of the State, as shown by the recent United States Census. This irrigated area is served by 3 305 miles of main canals, and 2 361 separate diversions. The territory surveyed is scattered over 500 different townships.

The streams, including all their tributaries, which have been surveyed, in addition to those mentioned above, and on which the adjudication proceedings are well under way, are given in Table 2.

TABLE 2.—Summary of Survey Maps.

Stream.	No. of diversions.	Length of main canals, in miles.	No. of townships covered.	Acres irrigated.
Crooked River	152	259	37	19 854
Rogue River	324	463	46	13 715
Umatilla River	259	397	38	15 902
Willow Creek, Malhuer Co.	175	219	16	9 412
Powder River	791	879	52	85 112
Silvies River	208	228	24	64 130
Lost River	37	25	13	5 420
Bechdoldt Gulch	2	4	2	148
Cherry Creek	3	5	1	225
Totals	1 951	2 479	229	213 918

New Rights Granted.

A total of 2 084 applications for permits to appropriate water have been filed with the State Engineer during the past three years. Of this number, 1 024 have been approved, becoming permits, and more than 400 have lapsed or have been cancelled from the records. These permits involve the construction of 2 500 miles of canal and 163 reservoirs, at a total estimated cost of $32 250 000. These works, if con-

structed, will ultimately irrigate 723 100 acres and develop 111 400 h.p. (theoretical). This area is 37 100 acres in excess of the total area which is now being irrigated, according to the recent United States Census. This remarkable showing is due perhaps largely to the confidence of capital in the new law, and it is the small investors, rather than the large ones, who have produced so startling a result in the aggregate.

The total fees collected to date, by the State Engineer's office, under the provisions of the new law, amount to $41 388.60, which is approximately $11 400 in excess of the appropriation for the department during this period. These fees are in themselves a reasonable guaranty that the proposed works will be constructed in accordance with the permits issued.

Protection of Rights.

There has been only one year's experience in the distribution of water by the water master under decrees of the Board of Control. In every case the people have been well pleased with the results, as compared with the old method of enforcing rights by injunction proceedings before the Courts. The law is a success only in so far as the administrative authorities can distribute the water in accordance with the decree, and at the times when it is needed.

Riparian Rights.

Approximately, 2 000 claims to water have been filed in these adjudication proceedings before the Board. All but four have been based on the theory of appropriation and use. These four claimed as riparian owners, but proved rights by appropriation. To the water user, therefore, the question of riparian rights is a dead issue in Oregon.

The riparian owner, under the decisions of our Supreme Court, cannot secure a decree of any definite quantity of water. He has only a reasonable use in common with all other riparian owners. The water to which he is entitled, therefore, is a fluctuating quantity, impossible to define or to protect. What are riparian lands one day, may be enlarged the next by purchase so as to extend even beyond the watershed of the stream. It is only a matter of time when our Supreme Court will declare that this common law doctrine is not applicable to conditions in Oregon, and therefore never has been the law of this State. Recent decisions tend strongly in this direction. "The rules

respecting the tenure of property must yield to the physical laws of Nature, whenever such laws exert a controlling influence."* The necessities of the people require that water shall be diverted for irrigation and other beneficial uses, owing to the peculiarities of soil and climate.†

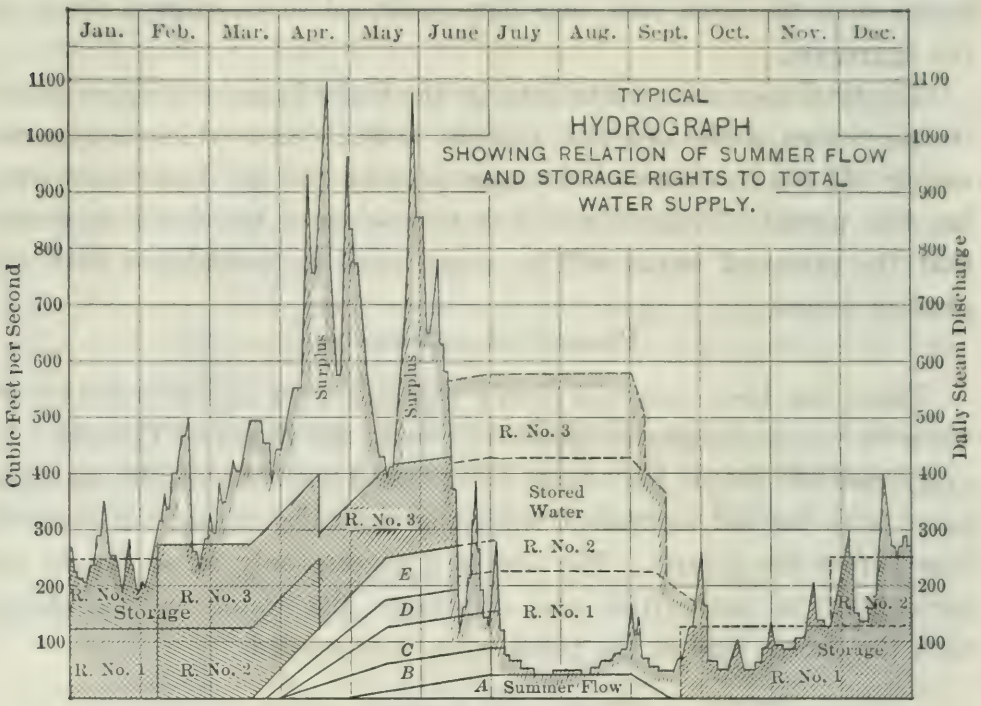


FIG. 2.

The riparian owner must conform to the State laws relative to the appropriation of water. Failure to do so would destroy the system of water titles and make State protection impossible for the water user. Lack of State control will result eventually in unending litigation and community quarrels over water. The new law amounts to an appropriation of all unappropriated waters, for disposal under the new system, and not otherwise. The preservation of the peace and safety of our citizens makes necessary the upholding of this system under the police power of the State.

WATER MAPS.

Water can be measured and shown graphically on a map with as great a degree of accuracy as can land areas. The water map (Fig. 2), or hydrograph, shows at a glance the total volume of water discharged

* Yunker *vs.* Nichols, 1 Colo., 551, 72.
† Walsh *vs.* Wallace, 26 Nev., 229; Twaddle *vs.* Winters, Nev., 85 Pac., 280.

by a stream at a given point; it also shows the discharge for each day in the year, the extent of the regular flow and storage rights which have become vested, and the surplus water available for new appropriations. On it can be shown graphically the character and extent of all diversions, whether from the stream for direct use (Ditches *A, B, C, D,* and *E*), or for storage (Reservoir Feed Canals Nos. 1, 2, and 3). The stored water will be released during the summer, and the extent to which the regular flow of the stream will be supplemented can also be shown graphically. Having careful records of daily stream discharge extending over a series of years, and accurate records of vested rights as determined by the Board of Control under the new system of water titles, it is a simple matter to construct hydrographs which will show with great clearness and certainty the volume of surplus water available for new appropriations. The new law provides a definite method for initiating rights to such surplus water, and grants full protection to such rights during the period of construction.

The Oregon system of water titles is of value because it is complete. Full power for determining old rights, for granting new rights, and for the protection of all rights, is concentrated in one Board. The records are uniform, in simple tables by stream systems, assembled in one central office, where they are thoroughly indexed and digested for quick reference. Every step is taken in harmony with a preconceived plan, and with the purpose of bringing order out of chaos.

The information as to the total water supply is of primary value in encouraging new enterprises. The State is co-operating with the U. S. Geological Survey in the joint expenditure of $56 000 annually on stream gauging work, the making of river profiles in aid of water-power development, and the making of topographic maps. Records of stream flow and of vested rights to water are absolutely necessary for the proper design of new works involving the use of water. We are beginning to realize that it is useless to expend money in the construction of hydraulic works unless water can be delivered with certainty at the time when it is needed. For these reasons, definite State control of all public waters has been undertaken in Oregon.

Interstate Rights.

Just as the lack of a definite system of titles to water within a State depreciates the value of works constructed for the utilization of

water, promotes litigation, strife, and occasionally bloodshed among its citizens, and discourages settlement and new development, so the lack of some interstate or Federal control as to interstate waters affects the territory adjoining such streams in either State, irrespective of whether one or both States have adopted a modern system of control for waters within their borders.

Already interstate troubles are beginning to loom large for the State of Oregon, and it is probable that almost every other State in the Union is similarly affected to a greater or less extent.

A statement of these difficulties will doubtless be of value in showing the need of some legislation in the near future relating to interstate waters.

The United States Reclamation Service has appropriated all the waters of Klamath River and its tributaries in Oregon for the irrigation of several hundred thousand acres of land in Oregon and California. The river, a short distance below, flows into California, where its waters are of value for power development. Power rights in the lower State may in the future be seriously affected by the diversion of water above for irrigation; but a more immediate cause of difficulty is the construction, by the United States, of a dam, at the outlet of Clear Lake in California, which will require three years for the reservoir to fill, when the waters will be turned backward, over a divide, into California, when they have heretofore flowed northward into Oregon and have been used largely for the irrigation of lands in Langells Valley.

Farther to the east, in Lake County, the State of Oregon has under consideration the reclamation of 100 000 acres of land in Warner Valley, under the provisions of the Carey Act. The water for this project will be derived largely from California and stored in a reservoir just over the line in Oregon. The remaining water will come from Nevada drainage. With this project completed, subsequent diversions in California and Nevada may seriously injure prior rights in Oregon.

A 3 000-acre private project on the East Branch of Lost River will depend on water from California, with storage on a tributary, which must be conveyed through California territory to reach the land in Oregon. There are many smaller projects along the southern boundary line which are more or less complicated.

Between Oregon and Idaho several complications now exist. Under

the Carey Act, this State has undertaken the reclamation of 50 000 acres of land in Jordan Valley, the water to be stored on Jordan Creek, just over the line in Idaho. Water can be diverted from this stream just above the reservoir and carried through a short canal in Idaho to Sucker Creek, down which it can be dropped to a reservoir in Oregon. Plans are under way to use this water, supplemented by Sucker Creek, for the irrigation of from 30 000 to 50 000 acres in Idaho.

Diversions from Snake River, in Idaho and Wyoming, in recent years have lowered the water surface so much near Ontario, Ore., that the Wilson Ditch cannot secure its accustomed water supply. Water-power permits granted by Oregon on the Snake River, later, may embarrass Idaho seriously in making diversions for the higher use, irrigation. Diversions from Snake River have affected navigation noticeably on the stream below Lewiston, in Washington.

A large canal has been constructed, tapping the Walla Walla River in Washington; it crosses into Oregon, and irrigates a large area in both States. This stream in Oregon has been in almost constant litigation for thirty years, and a case, started six years ago, and involving more than 400 parties, has just reached our Supreme Court.

Litigation is now in progress on Goose Creek in Idaho, involving Nevada waters passing through Utah. The most complicated case known to the writer is that of Bear Creek, which rises in Utah, flows through Colorado and Idaho, and terminates in Utah, and along which there are extensive diversions in each State and constant trouble over water.

These illustrations should be sufficient to show the real necessity for some legislation. In the Eastern States, no doubt, many similar difficulties have occurred over diversions for municipal water supplies, canal supplies, and for other uses peculiar to that section.

Character of Legislation.

We have paid dearly to learn that county control of waters was impossible. After the sacrifice of much valuable time and money, the destruction of interstate rights, the engendering of strife and bad feeling between States, and the retarding of development in the territory tributary to interstate streams, we may learn that State control of all waters within its borders is just as impossible as county control.

Streams do not follow county lines any more than they follow State lines. Such a division is impossible. We should be able to store water in Wyoming for the irrigation of lands in Malhuer County, Oregon, using the Snake River as a canal through Idaho, just as we can store water at the head of the John Day River in Grant County to be conveyed under State protection through other counties, for use at the mouth of such stream in Oregon. It should be possible to store the flood-waters of the Colorado River in the available basins in Colorado, where no use is possible in such State, and convey this water, under United States or interstate protection, for use in Southern California. The fullest development of streams will lag until such time as some legislation is provided affecting the determination, the granting, and the protection of interstate rights. We cannot expect the United States Supreme Court to work out and adopt some administrative system adequate to the needs in this respect. This is a problem for the legislative branch of the government.

The National Irrigation Congress at Boise, Idaho, in 1906, appointed a commission, of which the writer was a member, to investigate the subject of interstate rights and report the next year at the Sacramento meeting. The views of this commission, as herewith presented, are rapidly being accepted by a growing number of people as the only solution of this vexing problem:

"In view of existing conditions which place the burden of the admininstration of water rights upon the different States, your committee believes and recommends, as do a great many of those answering the interrogatories [replies to circular letters], that the different States affected by interstate streams take some steps looking to a more uniform system of laws affecting water rights, making those laws of each State agree as nearly as possible with those of sister States, and that there should be a general overhauling of all the laws of the arid States as to the matter of water rights, to the end that litigation as to this character of property may be reduced to a minimum, and that the greatest good may come to the greatest number in the matter of the use of water for irrigation.

"While it is true that, in the administration of water rights upon interstate streams by the different States, the right of appeal to the Federal Courts exists, that remedy is expensive, slow, and unsatisfactory. A decision of a Court, once rendered, remains fixed, and only settles the particular questions involved in that case, while the conditions surrounding irrigation on either side of the State line are

constantly changing, and the use of water for irrigation is rapidly growing.

"If there is to be any protection of priorities across State lines, it should be by a Federal administrative system corresponding in character to that needed for the establishing and protection of rights within a State.

"Respectfully submitted,
"ELWOOD MEAD,
"FRANK FREEMAN,
"MORRIS BIEN,
"JOHN H. LEWIS,
"Committee."

After six years of further consideration of this important question, the writer is of the opinion that such Federal administrative system should not be confined to the Western States, but should apply to all territory over which the United States has jurisdiction, and embrace all interstate waters and navigable waters, including the non-navigable tributaries of such navigable streams.

STATE OR FEDERAL CONTROL.

This brings us to a consideration of the question of State or Federal control of waters.

As to navigable waters, there appears to be no question as to the authority of Congress to enact any measure which, in its judgment, is necessary to preserve the navigability of navigable streams, even against any State action.

The Supreme Court of the United States, in the case of United States *vs.* Rio Grande Dam and Irrigation Company,* said:

"Although this power of changing the common law rules as to streams within its dominion undoubtedly belongs to each State, yet two limitations must be recognized: First, that in the absence of specific authority from Congress a State can not by its legislation destroy the right of the United States, as the owner of lands bordering on a stream, to the continued flow of its waters; so far at least as may be necessary for the beneficial uses of the Government property. Second, that it is limited by the superior power of the General Government to secure the uninterrupted navigability of all navigable streams within the limits of the United States. In other words, the jurisdiction of the General Government over interstate commerce and its natural highways vests in that Government the right to take all needed measures to preserve the navigability of the navigable water-

*174 U. S., 690-703.

courses of the country even against any State action. It is true there have been frequent decisions recognizing the power of the State, in the absence of Congressional legislation, to assume control of even navigable waters within its limits to the extent of creating dams, booms, bridges, and other matters which operate as obstructions to navigability. The power of the State to thus legislate for the interests of its own citizens is conceded, and until in some way Congress asserts its superior power, and the necessity of preserving the general interests of the people of all the States, it is assumed that State action, although involving temporarily an obstruction to the free navigability of a stream, is not subject to challenge."

If, therefore, under the interstate commerce clause of the Constitution, Congress has direct power to exercise control over navigable waters, as far as their navigability is concerned, it must follow by necessary implication that Congress can also exercise such control over the non-navigable tributaries of such stream as is necessary to preserve such navigability.

As pointed out previously, the diversions from Snake River, for irrigation, domestic, and other uses, have diminished the flow in the lower river to such an extent as to affect navigation materially. Likewise, diversions from the tributaries of the Willamette River, which is wholly within the State of Oregon, will affect navigation, by reducing the volume of water, and making dangerous certain rocks on shoals which would not otherwise be dangerous to navigation.

If private capital, or even the Reclamation Service, a branch of the United States Government, should construct reservoirs to store winter waters, and divert the regular summer flow to such an extent as to injure navigation, such diversions could doubtless be enjoined by Congress, or by appropriate action before the Supreme Court. It is high time that an end be put to this system of government by injunction, and that definite legislation be enacted so that capital can invest with safety, under proper Governmental sanction and supervision, in those large undertakings which will bring prosperity and happiness to the present generation.

"As to non-navigable streams within the State, which are not tributary to a navigable stream, it is difficult to find any possible grounds for the theory of State control. Where the United States has disposed of its public land, it has either transferred the water with it to the new owner, or has retained its right to it. There is no opportunity for the intervention of a State claim."*

* Morris Bien, U. S. R. S.

In the original thirteen States, the waters are either the property of the public, or are vested in the riparian owners.

It is true that in 1866 Congress recognized the rights of appropriators, and in 1877 dedicated waters on the public domain to appropriation and use, but no where do we find a definite grant of waters to the State. The nearest approach to a grant to a State occurred in the approval by Congress of the constitution of the State of Wyoming, containing these words, "all waters within the boundaries of the State are hereby declared to be the property of the State." This constitution was "accepted, ratified, and confirmed by Congress." In the absence of the words of an express grant, it is doubtful if the mere approval of a voluminous document will ever be construed by the Supreme Court to be a grant of public waters.

The case of Kansas *vs.* Colorado* apparently abolishes riparian rights as to the waters of all States, and at least as to all interstate streams. In this case:

"The citizens of Kansas insisted that large quantities of water were being diverted from the Arkansas River by the inhabitants of Colorado, a large number of whom were claiming as prior appropriators and diverting the water to non-riparian lands, as against Kansas, a riparian proprietor. In Kansas, the modified doctrine of riparian rights prevailed, while in Colorado, prior appropriation was and is recognized as the governing doctrine. The Court refused injunctive relief, and dismissed the bill, stating in substance that if the riparian doctrine should prevail in Kansas as against Colorado and against the non-riparian users whose rights were involved therein, Oklahoma and its citizens lower on the Arkansas River might invoke the same rule in opposition to both citizens of Kansas and Colorado, to their great injury, which doctrine, it is observed, would be ruinous in its effects. The Court, in dismissing the bill, indicated that no injunction would lie until a more substantial injury could be shown, and at the same time found that the interference by a large number of appropriators above in the State of Colorado, materially depleted the flow to the riparian lands of the plaintiff."†

There can be no misunderstanding of the clear intent of the Supreme Court to permit of the diversion of water in an upper State by non-riparian proprietors, even though such diversions materially deplete the flow to riparian lands in the lower State. From the definition of a riparian right, as heretofore given, we observe that it is a property right inseparably annexed to the soil, and passes with it, not

*206 U. S., 46.

† Hough *vs.* Porter, 51 Oregon, 409.

as an easement, or appurtenance, but as a part and parcel of the land. It is, therefore, a vested right to private property, which, under the Constitution of the United States, the owner cannot be deprived of without due process of law, nor taken for public use without just compensation. We must assume that this recent, able, and carefully considered opinion of the Supreme Court is not in violation of this section of the Constitution, and that it is applicable alike to all States. Therefore, we cannot escape from the conclusion that the common law doctrine of riparian rights, which has been assumed as the law by the Courts in many of the Western, and doubtless all of the Eastern States, is not the law, and therefore never has been the law, notwithstanding many decisions of the State Courts to the contrary, at least as to interstate waters. If it is not the law as to interstate waters within the State, it is difficult to conceive of any theory on which the State Courts could maintain such doctrine for those streams which are wholly within the State and are not tributary to navigable waters.

In summing up this case, the Court said:

"Regarding the interests of both States and the right of each to receive benefit through irrigation and in any other manner from the waters of this stream, we are not satisfied that Kansas has made out a case entitling it to a decree."

By this language, we believe that the Court had in mind, not alone irrigation, but all possible uses that water could be put to in the upper State, in any part of the territory under the jurisdiction of the United States.

The Courts of many of our Western States, long prior to this decision, had reached this same conclusion, but by a different process of reasoning.

"When the question of water rights was first considered in the State of Nevada, the Court held that the patentee of the government succeeded to all of its rights, and among these was the right to have the water of a stream, theretofore diverted, returned to its natural channel. But this case was over-ruled in Jones *vs.* Adams. And in Reno *vs.* Stevenson (20 Nev., 269) it was unequivocally declared that the common law doctrine of riparian rights was unsuited to the conditions of that State. In the leading case of Clough *vs.* Wing (2 Ariz., 371) it is said that the common law, so far as the same applies to the use of water, has never been, and it is not now, suited to conditions that exist here."*

* Wiel, on Waters, p. 188.

This same line of reasoning can and should be applied to every State of the Union, so that the atmosphere will be cleared for the enactment of a comprehensive system of titles to water for the United States as a whole, for in every State it is necessary to divert water for some useful purpose. If precedent and antiquity determines the law, then this common law doctrine should give way to the civil law of Rome, which is based on prior appropriation and use, and can be clearly traced to the earliest history of ancient times.

If, therefore, riparian rights are not considered as vested rights by the Supreme Court, then the inevitable conclusion must be that water is the property of the public, and that Congress, under the domestic tranquillity and general welfare clauses, has power to adopt a comprehensive system of titles to water which will obviate the present turmoil and confusion as to water rights, and will promote the general welfare of the Nation in proportion as the Oregon system outlined herein has promoted the general welfare in Oregon.

Equitable Apportionment of Benefits.

In summing up the conclusions in the case of Kansas *vs.* Colorado, the Court, after stating that Kansas had not made out a case entitling it to a decree against Colorado, said:

"At the same time it is obvious that, if the depletion of the waters of the river by Colorado continues to increase, there will come a time when Kansas may justly say that there is no longer an equitable division of benefits, and may rightfully call for relief against the action of Colorado."

The question naturally arises as to what constitutes an equitable apportionment of benefits, and who has the necessary authority to decide as to this matter.

There is no question as to the ability of the Supreme Court to decide this question in any case properly before it. To take to the United States Supreme Court every case which is now in controversy, is entirely out of the question. The time and expense involved would compel those suffering injury to endure the wrong rather than appeal to the Court. Such an appeal in most cases is entirely beyond their resources.

If in the Kansas-Colorado case, the Court refused to grant to the plaintiff the required relief, after consuming eight years' time in the

examination of 347 witnesses, the taking of 8 559 typewritten pages of testimony, and the making of 122 exhibits, at a cost of about $200 000, it might be a fair question to ask one who believes in this system of government by injunction, how much longer must Kansas suffer before making a second attempt to secure relief, and what would be the estimate of the cost, in time and money, to complete such second attempt.

This system of government by injunction is disastrous to progress. Let it be assumed that the State of Oregon brings suit against the U. S. Reclamation Service to enjoin the storage of water in its Clear Lake Reservoir, recently constructed in California, and wins; it is easy to picture its effect on future development along our State border. No one would dare to invest without prior sanction from some Governmental authority.

In the absence of any specific guidance in such decree, it can only be assumed that Congress must have power to prescribe by law what shall constitute an equitable division of benefits as to interstate waters. Any other view would make the decree ridiculous, as the Supreme Court could not expect that every interstate water problem would be submitted to it for decision. This is an administrative matter; the Court has decided the law, and it remains only for Congress to provide suitable machinery for carrying the law into effect.

If Congress should decide that the enforcement of the doctrine of priority of appropriation and beneficial use, subject in each case to the consideration by the administrative authorities as to the public interest and welfare, should constitute an equitable apportionment of benefits, it is conceivable that the Supreme Court would uphold such act, on the ground that the question involved is of a political nature and therefore not subject to review by the Court.

State control was not undertaken in our Western States until the patience of the water users had been exhausted so thoroughly by the exasperating delays and expenses of litigation, and the inability of the Courts to distribute the water through injunction proceedings, that they took the law into their own hands, with the result that many murders were committed. It is conceivable that war between States over the proper division of interstate streams may result, before Congress can find time to exhaust thoroughly all the possibilities for debate in this most complicated and involved subject. Decisions can

be found sustaining almost any view. It is easy, therefore, to get hopelessly involved in the details of this subject unless one keeps clearly in mind the ultimate objects to be attained, and proceeds on the theory of the greatest good to the greatest number.

The writer has thus analyzed the question of State and Federal control with respect to navigable waters, including their non-navigable tributaries, non-navigable streams wholly within the State and not tributary to navigable waters, and lastly, the question of interstate streams. He is led to the conclusion that Congress, under the present Constitution, can provide by law a definite system of water titles for the Nation, similar to that adopted in Oregon.

State's Rights.

The writer is aware of the criticism that will be urged against this doctrine by the advocates of State's rights, but he is unable to reach any other conclusion which will harmonize with the leading cases and the statutes as we now understand them. It may be of value, therefore, to inquire as to what extent, if at all, an individual State will suffer through the adoption of a National system of water titles.

Prior to the Kansas-Colorado decree, the State of Wyoming, which claims to exercise absolute control over property in water within its borders, would have had good ground to oppose such a system. The State is located on the Rocky Mountain Divide, and all waters originating within its borders flow into adjoining States. Without such National administrative system to restrain over-appropriations in Wyoming, to the detriment of prior appropriators in adjoining States, Wyoming may be considered to have an advantage, being the upper State, and therefore needing no protection from appropriations in adjoining States, while, on the other hand, with a National administrative board to pass upon interstate permits prior to construction, investments could be made without fear of being enjoined by lower States subsequently, and thus, through rapid and safe development, Wyoming would doubtless receive greater benefits from this new system than under the present lack of system. Just as the upper appropriator on a stream needs no protection from lower appropriators, so Wyoming needs no protection from other States; but few States are similarly situated.

If, by such Congressional legislation, the various States which have not yet assumed control of their water resources could be induced by outside pressure to adopt uniform State laws, such States, as well as the country as a whole, would be greatly benefited.

Congress having the power to legislate as to waters, it is difficult to understand how such a body, made up of representatives from the different States, could enact any law which would do great injury to any State or any section of the United States. The control and distribution of waters within the State will be left to the State's administrative authorities. The control of navigable and interstate waters, say in the Columbia River Basin, will be left to the State administrative authorities of this region in proportion as each State is interested, subject, of course, to the general laws, and the rules prescribed thereunder. The Federal administrative board could be composed of all State water authorities meeting annually in convention, or by a smaller board composed of representatives from different sections of the United States.

With the establishment of a National administrative system corresponding to that which has been proven a success in Oregon, Wyoming, and a few other Western States, a comprehensive plan for the development of our rivers in aid of navigation could be outlined and carried to completion without fear of entanglement over interstate rights. The public welfare as to any particular project on an interstate or navigable stream could be considered by the Federal authorities, and, upon approval, construction could proceed with certainty. Such Federal commission should have authority to determine and record all existing rights to water in those States where no such records are available, and should consider the relative value of unappropriated waters to the different States for domestic, irrigation, power, and navigation purposes, and have power to refuse any permit where the proposed use was a menace to the safety or welfare of the public.

Such a National system of water titles, with adequate administrative machinery to make it effective, would be of as great advantage to the Nation as the State system is to the individual State. The respective States would derive the same benefit from such system as the individual water user within the State does from the State system. We have priorities in rights to interstate waters, the same as for State waters. We have strong, aggressive, and litigious States endeavoring

through litigation to get some undue advantages over a weaker and upper State in the matter of interstate streams. We have upper States taking more than their rightful share of the interstate water supply, to the detriment of citizens in a lower State, and these citizens are compelled to suffer because of the lack of interest on the part of their State Government, or the National Government, or some other political or financial reason beyond their control. In other words, the "water hog" is not confined alone to State streams, but is beginning to make himself felt on the larger interstate streams. Here he is so intrenched behind technicalities of the law, and ignorance, due to lack of information as to the effect of numerous diversions on long rivers, that we fail to recognize him. Just as some people within the State, who have an undue advantage in water appropriations, oppose the enactment of State water laws, so we will have States opposing a National water law. Such law would be of no value if it did not restrain diversions in some States for the advantage and protection of others.

The argument of State's rights will be used effectively for a time against the enactment of a Federal water law. The individual State will, in the opinion of the writer, be benefited by such a law, and the Nation, as a whole, receive very great benefits.

Water-Power Policy.

The attitude of the United States toward the policy of water-power development is of particular importance to the Western States, for it is in the West that the greatest amount of undeveloped water power is located. The United States controls the major portion of this power through National Forest Reservations and the withdrawal of all vacant lands valuable for water-power sites. The State, in the absence of Congressional legislation, exercises control over the water, but it is impossible to use this water for power development without access to the stream. Right of way for such purpose cannot be secured without the consent of the United States, as condemnation cannot be resorted to.

Power sites in the National Forests can only be developed under revocable permits, and this is about as serious an obstacle to development as a complete withdrawal. In the streams of Oregon, the U. S. Geological Survey estimates that 3 317 000 h.p. are undeveloped. It

is of great importance to the development of the West that these restrictions in the path of development be removed as soon as possible. As the land is the property of the United States, there is no question as to the power of Congress to enact necessary laws, and such legislation need not await necessarily the adoption of a comprehensive system of water titles for the Nation.

That all power sites remaining on the public domain should be retained permanently by the Government, and be made to yield a material revenue, was suggested recently by the Commissioner of Corporations, at Washington, D. C. He states, further, that:

"Fuel-power will substantially fix the price of all power, because there is practically no considerable area in the United States where water-power can supply the entire demand for power. Of course, water-power cannot be sold above the price of fuel-power. On the other hand, if the price of water-power be fixed by law below that of fuel-power, not all the community, in most instances, can be served with cheap power, and an unfair discrimination must result."

Such policy would seriously injure the West, where water-power is plentiful and coal is dear. It is just as reasonable to argue that cheap coal development of the East should be made to yield a material revenue to the Federal Treasury, so that the West, where coal is dear, may not suffer a material handicap. Besides, such a policy is entirely antagonistic to the conservative theory of preserving to posterity a fair share of our exhaustible coal, oil, and timber supplies.

The views of Secretary Fisher should prevail, as approved by President Taft, and outlined in his message to Congress of February 2d, 1912. That the revenue derived from the development of water-power should be sufficient only to cover the cost of administration of that particular department, any surplus to be expended by the General Government for the improvement of the stream, and the benefit of the local community where the water-power site is located.

A more desirable policy, in the opinion of the writer, would be for the States, in co-operation with the United States, to develop this power and supply it at cost plus interest. There is no organization which could secure money at a lower rate of interest than these agencies, and, besides, the greater the scale of the development, the cheaper the original and maintenance costs.

The United States is now engaged in the development of hydro-

electric power, and is selling it at cost to municipalities in the vicinity of reclamation projects for light, heat, and other purposes. The winter rate, on the Minidoka Project of the U. S. Reclamation Service in Idaho, is so low as to permit the use of electricity for the heating of dwellings in competition with coal. The rate for heat, the writer is informed, is $0.50 per kw. per month, with $0.50 allowed to the private company for distributing the power within the municipality. If the States should join in furnishing funds, and in extending this organization, which has already made a success in this enterprise without the least suspicion of scandal, it would only be a few years until the entire United States would be covered with a network of transmission lines. This would be conservation on a grand scale.

For the encouragement of irrigation by pumping in the West, such a policy is indispensable. The cost of electrical power, under the most favorable circumstances, including interest, depreciation, and maintenance, for a pumping project, is very high, and a farmer is justified in undertaking reclamation by this method only under the most favorable conditions. Add to these obstacles the chance of paying an increased rate for his power at the end of the usual short-time contracts with power companies, and we have such unfavorable conditions as will compel much of our territory, which could otherwise be reclaimed, to remain arid indefinitely. Irrigation by gravity flow did not become a success in the West until the land and water were united in one by laws making water appurtenant to the place of use. Many communities under common carrier canals became so discouraged, as they watched the charges for water gradually increased in proportion to their ability to pay, that they abandoned their homes. This led to a reform in the water laws.

In 1870, 49% of the population of the United States were farmers. In 1900, only 29% were farmers. This is a most serious condition for our Government to face, if it is to endure. Some of the luxuries of life have come to be considered necessities. These must be supplied to the farmer at such prices as he can afford to pay. We are endeavoring to check this unhealthy trend by supplying good roads, telephone service, rural free delivery of mail, good schools, etc., for the farmer. The writer can conceive of nothing in addition to these which would make country life more attractive than the supplying of electrical

power to the farmer at cost. It can be put to almost immeasurable uses in the home, and on the farm generally.

Private capital will probably never be able to supply the farmer, owing to the relatively long transmission lines required for the limited market. Under State and Federal control, electric rates in cities could be lowered, even though the added cost of country service was assessed on all.

To prevent a threatened water-power monopoly, and to reduce electric rates, the Province of Ontario, Canada, created a Hydro-Electric Power Commission, in 1906, with full power to construct plants and to buy and sell power. This commission is now operating 281 miles of 110 000-volt main transmission lines and 180 miles of distribution lines, buying current from the Ontario Power Company, a private corporation at Niagara Falls, at $9 per horse-power per annum. The rates charged the municipalities to cover cost, vary from $18.10 to $29.50 per horse-power per annum, according to distance, and, at present, twenty-one municipal corporations are supplied with an aggregate of 30 500 h.p.

The two attempts of public developments of power and sale at cost, as here outlined, have been successful thus far. They should prove of value to the Nation in considering public control on a comprehensive scale.

Conclusions.

The distribution of water and the question of water rights are, at best, complicated matters. The present confusion seems to have arisen from the fact that heretofore we have left to the Courts the solution of all difficulties arising from the use of water, when, in fact, they should have been solved by the legislative and executive branches of our Government.

The Courts, having no knowledge of the practical difficulties involved in the dividing of streams for the different users, and approaching the problem only after investments had been made, to the injury of other investors, have established and maintained a system of law which is entirely repugnant to the diversion and use of water for any but the most primitive uses. Following sound rules of law which had been developed under entirely different conditions, the Courts have refused to the legislative branch the right to amend the Court-made law, on the ground that such statutes would be an inter-

ference with vested rights and in violation of the Constitution. Realizing the extreme necessity of diverting water from streams in the Western States, the Courts, in a few instances, have declared that what they had heretofore considered the law was in reality not the law at all. Now the Supreme Court of the United States, in a decision which applies alike to every State of the Union, appears to indicate that the Courts of all Eastern, and many of the Western, States have been mistaken as to the law of waters, and the way is now clear for the first time in many years for legislation. This legislation should be positive, and should approach the problem from the standpoint of the investor prior to undertaking development. The public interest should be carefully considered prior to the permitting of any new development.

All rights heretofore initiated in good faith for the application of water to any useful purpose should be determined, recorded, and thereafter receive the protection of the law, instead of each user being left to his own individual efforts.

The common error, made by a number of Western States which have attempted to legislate as to waters, should be avoided; that is, to assume that the determination of the facts (priority, purpose, period, place, and quantity), necessary for the establishment of a water right under the law, is a judicial matter, which can only be passed upon by the Courts. Such error would be fatal. These matters should be attended to by an administrative board which has complete jurisdiction of all matters relating to water. Every step thus taken will be in harmony with some preconceived plan, and the final results will be of correspondingly more value. Any error in law can be cured by the party injured through an appeal to the Courts. The lack of such appeals, from the administrative officers in Wyoming and Oregon indicates that but a few appeals will be necessary. If, in such system for the adjudication of old rights, the complicated legal rules of pleading and of evidence had to be followed, it is difficult to conceive the cost in time and money, and it is doubtful if a complicated adjudication could be carried through in an ordinary lifetime.

To illustrate, the leading case of Hough *vs.* Porter was before the Courts of Oregon for ten years, before a final decree was rendered. There were only forty-five parties to the suit, and an administrative

board could have secured all necessary information and decided the matter in one-tenth the time.

The State of Utah adopted a system of water titles in 1903, where the State Engineer was to make surveys and gather facts for the guidance of the Court in the adjudication of water rights. After nine years, and the expenditure of about $30 000 in the survey of the Weber River, involving about 400 ditch rights, not one right has yet been determined by the Court. This record is in striking contrast with the work of the administrative board in Oregon as outlined in this paper. In only a few years every early right in Oregon will be determined, and will receive the protection of the State's administrative officers.

The subject here presented relates more to the legal, than to the engineering, side of the water problem, and yet only the engineer can appreciate with full force the importance, to the country as a whole, of having definite and uniform State laws relating to water, supplemented by a National administrative system in harmony therewith. Therefore, he should take a leading part in the reform of State and National water laws. Streams do not follow State lines, and for this reason, these arbitrary political divisions must give way to the more enlightened policy, which is in harmony with the laws of Nature, of treating each stream system as a unit from its head-waters in the forests to its mouth on the coast.

DISCUSSION

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CLARENCE T. JOHNSTON, M. AM. SOC. C. E. (by letter).—This paper deserves some comment from those who are interested in problems of stream control. The writer is glad to find, here and there, an engineer who has had sufficient experience in such work to convince him that the questions which arise are of a legal nature only in an incidental way. In its fundamental aspects, the supervision of water resources is of greater public concern than the protection of land titles. Water and watercourses are generally considered as public property, and, therefore, the demand for an engineering administration has not been felt, except where streams have been used to a large extent, and particularly where water has been diverted from natural channels. As controversies have arisen, the Courts have been appealed to, and to-day, particularly in the West, decisions of all kinds are made. Volume after volume is published dealing with theories which have been developed and played with by attorneys, until they can no more be applied in practice than water can be diverted from a stream without affecting its discharge.

We derive our common law from England. With that law we brought the doctrine of riparian rights, which guarantees to every owner of land abutting on a watercourse the right to demand that the waters thereof pass his property "undefiled in quality and undiminished in quantity." This is the true riparian doctrine. It is not suited to a country where large volumes of water must be taken from the streams. Regardless of this and regardless of the experience of other countries, lawyers and Courts have tried to make it fit every climate and every natural condition. A simple doctrine, which suits a country where large rivers prevail and where actual diversions are of little importance, has been distorted and juggled with by the Courts until it is no longer recognizable. The "modified" doctrine of riparian rights, as we have it inflicted on us, is often a convenient screen for crimes committed against the public and in behalf of those claiming "vested rights." A perusal of the statutes of various States referring to riparian rights, and quite generally the laws pertaining to stream control, will disclose the existing situation to the student. Much has been done to protect vested rights, but reference is seldom made to public rights. The idea that the public ever obtains a vested right would never be gleaned from reading the statutes dealing with stream control in a majority of the States. This condition arises from the fact that the lawyer, representing those who claim many special rights and privileges, often reaches the Legislature where he exhibits marked ability in inserting such clauses as the following: "This chapter shall in no wise be construed as impairing or abridging any rights already vested in any person or persons, company, or corporation, by virtue

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of the law heretofore in force." After a training of this kind he develops an instinct which enables him to protect private property rights without study or deliberation. Should such a man be elevated to the bench, he displays the same tendencies, and some of our remarkable decisions may be attributed to Courts so constituted.

Streams and lakes are not like land, in so far as private possession is concerned. Land can be measured and privately controlled. Water is constantly shifting, and the supply changes every day. The public must bridge streams and provide harbors. The public must protect fish and provide for the safety of dams. We use water to-day only to lose it to-morrow when it runs on, a continuous blessing to the public. It cannot be owned privately, and no State should ever permit individuals or corporations to claim such ownership. It is plain that injustice would be worked should one nation have exclusive ownership of one of the great oceans. It is equally plain that injustice would result should one State control an interstate stream or lake. The same rule applies when one individual or corporation is permitted to assume control of any local water supply, yet the local public is often slow to act in its own defense. The engineer should appreciate these facts. He should be a leader in questions of stream control. Unfortunately, engineers have not assumed the responsibility that naturally belongs to their Profession. We may criticize the Legal Profession and the Courts—and possibly they deserve it in some degree—yet we must remember that the engineer is depended on for information; and further, that where engineers have studied these important problems and given the Courts the results, reforms favoring public control have begun to appear. Too many engineers have been blinded by what seems to them to be the best policy at the time for their employers who appeal to the Courts. Too many engineers fail to see that principles which insure justice to the public protect water users generally. The water rights of private users are best insured where public control is most rigid. Engineers have been as tardy as the Courts in recognizing this.

When the National Constitution was framed, the States ceded to the General Government the control of navigable waters. The Nation, as a matter of development and defense, must control navigation when necessary. Non-navigable streams remain in the possession of the States. The States have not properly administered the smaller streams which they own. It has been so apparent that streams and lakes are naturally property of a public character that, for a long time, particularly in localities of large rainfall, the necessity for public supervision has not been called to the attention of lawmakers. Regardless of the public character of streams and lakes, private interests have assumed to take possession here and there, and the claims thereby arising are still to be settled. The great questions relating to State

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administration of waters are yet to be determined. Some Court decisions would appear to be final and in favor of a private monopoly of public property of this kind, yet the end has not been attained in a single important case.

Because the States have been slow in asserting the doctrine of public rights in property which is essentially of a public character, we need not fear that the public has lost title in any way. Because some individual or some corporation has been using this property for a term of years, there is no reason to hold that the users have secured title thereto. It would seem that such users would owe the public a vote of thanks at least. Many troubles have been precipitated because the owners of riparian lands have been permitted to claim more than the true riparian doctrine would admit in itself. Because the General Government has not meandered every small stream—a physical impossibility—the subsequent patentee of riparian lands has assumed that he owns the land to the center or entirely across the channel of the natural waterway. Where, through ignorance of the naturally public character of streams, the Courts have seemed to confirm claims of this nature, an error has been made, but such errors will be corrected in time.

Water must be used by individuals and by corporations. The public should realize this and consent thereto. The public and all users should appreciate the difference between the use of water and the ownership of water. All uses should be of such character, and secured under such restrictions, that the authority and ownership of the public are manifest in every transaction and apparent to all concerned. When the time comes for the public to assume full control of its property, private interests should be recompensed according to the estimated cost of replacing the works constructed by them, and which, under the new order of things, are to be operated by the public. Valuation work of this kind has also fallen largely to the engineer. It is a new science, and engineers have not reached an agreement as to the principles which should be applied in determining just valuations.

Principles are of so much more importance than details of law, methods of procedure, or exact character of administration, that the writer does not feel inclined to discuss National, State, or even more localized control of streams. Under present conditions, there can be no question as to the responsibility of the General Government and of the States in work of this kind. The essential facts which should determine any question which may arise are of an engineering, rather than of a legal, nature. They are simple and, as a rule, not difficult to obtain where an engineering administration is provided.

The writer has followed the development of the public control of streams of the West for many years. Wyoming, under the able leadership of Elwood Mead, M. Am. Soc. C. E., took the first step which

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relieved the Courts from all initial proceedings in questions relating to the use of water. Only appeals from administrative officers go to the Courts in that State, and these appeals are so few that during the past twenty years they can be counted on the fingers of the two hands. During that time the State Administration has studied and determined more than 15 000 claims and, at the same time, has protected all public rights. As Dr. Mead's Assistant for a term of years, and as State Engineer for nine years, the writer was able to give some thought to the principles which should underlie the laws relating to stream control, and the following matter from his last report* to Governor B. B. Brooks contains a discussion of the elementary principles:

"Some reference should be made at this time to Sections 724, 725, and 726, Wyoming Compiled Statutes, 1910. These sections were added to the irrigation laws of the State by the Legislature of 1909. The bill before the Legislature was known as 'House Bill No. 66.' All who have studied this legislation agree that it represents the most important action of the law-makers of the State since the original statutes were enacted in 1891. While the purpose of the act is fresh in the minds of those who were in the Legislature and among those who prepared the bill, some public record should be made of its purpose so that when its provisions are interpreted, there will be no mistaking its object.

"The Act reads as follows:

"CHAPTER 58.

"*Water Rights.*

"724. *Water Right Defined.* A water right is a right to use the water of the state, when such use has been acquired by the beneficial application of water under the laws of the state relating thereto, and in conformity with the rules and regulations dependent thereon. Beneficial use shall be the basis, the measure and limit of the right to use water at all times, not exceeding, in any case, the statutory limit of volume. Water being always the property of the state, rights to its use shall attach to the land for irrigation, or to such other purpose or object for which acquired in accordance with the beneficial use made and for which the right receives public recognition, under the law and the administration provided thereby. Water rights cannot be detached from the lands, place or purpose for which they are acquired, without loss of priority. (L. 1909, ch. 68, Sec. 1.)

"725. *Preferred Uses Defined.* Water rights are hereby defined as follows according to use: Preferred uses shall include rights for domestic and transportation purposes; existing rights not preferred, may be condemned to supply water for such preferred uses in accordance with the provisions of the law relating to condemnation of property for public and semi-public purposes. Such domestic and transportation purposes shall include the following: First—Water for drinking purposes for both man and beast. Second—Water for municipal purposes. Third—Water for the use of steam engines and for general

* Biennial Report of the State Engineer to the Governor of Wyoming, 1909–10, pp. 17 to 29.

railway use. Fourth—Water for culinary, laundry, bathing, refrigerating (including the manufacture of ice), and for steam and hot-water heating plants. The use of water for irrigation shall be superior and preferred to any use where turbine or impulse water-wheels are installed for power purposes. (L. 1909, ch. 68, Sec. 2.) Mr. Johnston.

"726. *Change to Preferred Use.* Where it can be shown to the board of control under the provisions hereof, that a preferred use is to be made, the procedure for a change of such use shall embrace a public notice, an inspection and hearing if necessary by and before the proper division superintendent, a report of such superintendent to the board of control, and an order by said board. If the change of use is approved, just compensation shall be paid and under the direction of the board, proper instruments shall be drawn and recorded. (L. 1909, ch. 68, Sec. 3.)

"The act is plain in its terms. There may be some confusion, however, in the minds of those who have not followed the history of the development of irrigation law in this State. The constitution of the State, which has the approval of Congress, says:

" 'Section 1, Article VIII. The water of all natural streams, springs, lakes or other collections of still water, within the boundaries of the State, are hereby declared to be the property of the State.'

"This provision of the constitution has been discussed from various standpoints. The State has never held that the water which it owns should be disposed of for profit. It is not deemed wise to administer this natural resource for revenue. It is presumed by the law-makers of Wyoming that the water the State possesses is for the use of the people. However, the State has a greater responsibility. Wyoming has discovered and placed on the statutes of the State the first definition of essential principles that should govern the use of water. So that the State, not only owns the water and permits of its use without charge to her citizens, but the State sees to it that all users are given equal protection. One man cannot be permitted to expand his use without interfering with the rights of a community. The State early discovered the danger of permitting water or water rights to be treated as personal property. If water rights are owned separate from the land irrigated, or such other use as may be made of the water, then the personal element enters and there is no such thing as public supervision and no such thing as the protection of the community against the greed of a few who may be powerful or influential.

"We cannot review the decisions of the Colorado courts without feeling that they have failed to enunciate such doctrines as will protect the water user. For instance, we find in the case, 'Laramie County Res. Co. *vs.* People *ex rel.* Luthe,' 8th Colo. 614, the following definition of 'appropriation.' 'Appropriation is the intent to take, accompanied by some open, physical demonstration of the intent, and for some valuable use.' The court evidently understood that a diversion from the stream without actual beneficial use of water could not give the right to use water. It was manifestly plain to the court that there was something necessary in addition to a claim, which we assume to be the manifestation of the 'intent.' The court did not see that an investment in irrigation works can be protected without giving the

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investors ownership in water. It saw that a time must lapse between the filing of the claim, or the expressing of the 'intent,' and the use of the water. Hence the priority would date from the time the claim was filed. The company, unless disturbed by some agency, would secure title to water under that priority date providing it completed the irrigation works. There was no limit to the demands the company might make on the stream, except its financial ability in ditch and reservoir construction. There was nothing in the plans submitted to tell where the prospective beneficial use was to occur and absolutely nothing to protect those who were to be the actual water users. It might have occurred to the court that the State could by law provide that a company be given a certain time to show its good faith and upon the final showing being made, that it be given a certain time to sell interests in the irrigation works constructed. It might have occurred to the court that this would have enabled each water user to have obtained water rights dating back to the time the claim was filed in some office of public record; it would have enabled the company to have obtained a reasonable profit; it would have left the water rights in possession of the water users, each having an interest in proportion to the land reclaimed; it would have given the irrigation works to those who should for all time rightly be held responsible for maintenance. How the construction of irrigation works without any arrangements being made with prospective users or without any reference to the proposed beneficial use, can give the builders a right to the water is a problem that is too complicated for the average man to grasp. The court saw the danger ahead and it tried to do something to avoid it. The trouble was that the court gave the company everything in sight, under certain conditions. It is like giving a railroad an entire valley on condition that transportation facilities be provided. The railroad has a right to demand lands which it uses or is to use. The ditch company has this right. Water is more valuable than land. The railroad company is not entitled to all the land in sight, when it cannot use the land. The ditch company cannot use all the water it can divert. It has no right, therefore, to be placed in position where it can sell water which it has never acquired by beneficial use. A railroad company which secures an entire valley receives a rich reward. It can sell the lands to those who can cultivate them. This arrangement should be very pleasing to the railroad company. It was doubtless as pleasing to the ditch company to get title to water. It could sell this water to those who either had to buy it or face financial ruin. There is absolutely no reason why an irrigation company, or any individual, should have title to any property except the physical works it constructs or acquires. The court did not see this. It did not see how the water users could be protected under the priority obtained by the initial filing except by giving the company title to the water. This is so simple and it works out so easily where the States have an administrative system which manages streams for the benefit of the public, that any other plan would never be discussed if it were not that living examples of a dangerous type are so close at hand. The Colorado court decisions go a little further than to give everything to the company which builds the irrigation works. The rights of the consumer are discussed in a number of cases. This

admits of some intimation of rights that others, aside from the irrigation company, might hold. If the court, in the first place, had held that the plans submitted by the company must be based on good engineering and that specifications for every structure must be filed; if it had then compelled the company to designate the lands that were to be reclaimed and dedicated the water to the benefit of those lands, a foundation would have been laid that would have supported every principle that might be necessary to protect those that were to follow the company. There is a library of speculation over this subject of appropriation in the Colorado decisions—yet no two complete discussions are in entire harmony. All indicate that there is something that should be reached and in every decision this something is just beyond the grasp of the court. To prevent too much injury being done the water users, the State Legislature has given the county commissioners authority to regulate the price paid for water. Because the court could not fathom the problem before it, the future of every project is left with administrative officers, who may or may not be able to determine what price is fair. The county commissioners may be connected with the company and they may be water users under the system in question. It would seem much better to make the water rights attach to the land and establish the price which the water user is to pay for an interest in the irrigation works in the beginning. Mr. Johnston.

"Let us study an individual case so that its history may be approximately complete. The Colorado courts dealt out water, rather than water rights, with a lavish hand. The courts did not know what volume of water was necessary to satisfy with justice the needs of each claimant, and they did not know what principles should govern in the adjudication of rights. No person can arrive at any other conclusion after reading closely the court decisions of the State. For instance, a water claimant is given a right to 35 cubic feet of water per second. The court does not say that this water belongs to the land or that the water right belongs to the land, but the person is given a kind of title to so much water. As the country grows the person so favored finds that the court has given him a large volume of water that he has never used. He therefore applies for a change in point of diversion. This may be opposed in the courts, but he generally wins out. He then proceeds to use this water on new lands, water that others have for years past been using. Those who are injured may be able to protect themselves if they have means. If not they must suffer the consequence. This person, who is so fortunate to get possession of water that he has never used, then proceeds to use it in another way that injures the community still further. Because the court has given him 35 cubic feet of water, he holds that he can store all of the excess that he does not need in the irrigation season for direct application from the stream. This is a happy thought. In the meantime possibly enough reservoirs have been built to store all of the excess flow of the stream. This makes no difference to this fortunate person. He stores, for his own use and for speculation, 35 cubic feet of water per second, except during the irrigation season, thus depriving other persons of his community of water that they have been using.

"He has probably secured the 35 cubic feet of water per second when

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he only irrigated 160 acres of land. Two to two and one-half cubic feet per second during the irrigation season would be ample for this area. Even should he be able to apply three cubic feet per second, he would have left 32 cubic feet per second for speculation during the irrigation season. This would supply water for 2 000 acres of land and be worth $80 000.00 at least. By storing 35 cubic feet per second for nine months he would impound 18 900 acre-feet of water. If a fourth of this, or 4 725 acre-feet of water are lost by seepage and evaporation, he would have left 14 175 acre-feet of water for further speculation. This would irrigate fully 6 000 acres of land and be worth $240 000.00. The court, therefore, through ignorance, and without considering the rights of any person except the claimant, gives him $320 000.00 worth of water that he has never used and which has been used by others. This value of $320 000.00 cannot be given to one claimant without taking it from another.

"What is the situation in Wyoming? An agricultural community begins to develop. Each applicant for water rights obtains permits which describe lands to be irrigated. The water rights are finally adjudicated and dedicated to the use made. The water used for irrigation belongs to the lands reclaimed. It remains attached to that land. When the land is sold it goes with the land. The law places a maximum limit on the use that can be made—one cubic foot of water for each seventy acres of land irrigated. The public must see to it that waste does not occur. This maximum limit cannot be exceeded at any time and the use is restricted to such volume less than the legal limit as can be beneficially applied to the lands to which the water right attaches. There is no way to expand the use. There is no way by which an individual or a company can speculate at the expense of a community. All rights are matters of record and there can be no serious cause for dispute.

"Because certain lands have water rights direct from the stream during the irrigation season does not mean that these lands are entitled to water throughout the year. The first reservoir built in compliance with law secures the first right to store water to its capacity. This reservoir can be filled at any time that the stream furnishes a supply that is not used. The second reservoir then has its turn, and so on. Reservoirs store water—not water rights. Ditches carry water—not water rights. Water stored in reservoirs simply augments the supply during the season when needed and this water leads to the perfection of water rights which belong to the lands reclaimed and to the other uses made, the same as though the stream supplied an ample volume during such seasons.

"Section 724 states that 'beneficial use shall be the basis, the measure and the limit of the right to use water AT ALL TIMES, not exceeding, in any case, the statutory limit of volume.' This means that when the community needs protection the public control must be such as to limit all users to the volume that can be beneficially applied. The certificates of appropriation, issued by the Board of Control, describe the lands to which the water right belongs and specifies what the maximum allotment of water is to be. It also says in conformity with the law that the use shall consist of the application of such part of the maximum allotment as can be beneficially applied.

"Water is the property of the State and no charge is made for its use. When water is used for irrigation or for any beneficial purpose for a term of years, the user should feel thankful to the public that conditions are such that he has been able to do business without being subjected to a special tax on the water. Because the public has enabled this to be done, is there any reason why any person or company should be given the additional right to sell the water or water right he has been enjoying? The community cannot be injured as long as the use continues as it has always been, but that use cannot change without having some effect on the rights of others. Because the last sentence of Section 724 states that water rights cannot be detached from the lands, place or purpose for which acquired without loss of priority, it is held by some that the act is unconstitutional. Under the constitution cities and towns can condemn water rights for municipal purposes. When a city or town needs water it gets it and always gets it regardless of priority or the character of the right. Under this same act, municipal uses are made preferred uses. It may often serve the purpose of a town to condemn a water right that belongs to some land not enjoying an early priority. For instance, the early rights may all be near the source of a stream. This is an ideal condition. The town may be twenty miles below the lands having these rights. There may be irrigated lands near the town that can be condemned to furnish a water supply for the municipality. The water right obtained by the city or town under such circumstances cannot affect the early rights at the head of the stream, yet the town has secured an adequate supply. Should it be necessary to condemn one of the early rights at the head of the stream, this could be done, at any time later. The idea is this—nothing should be done by the State to give an added value to the early rights. There are so many conditions affecting the flow of a long stream, that cities and towns can often be supplied without interfering with the early irrigation development. A town might buy an adjoining farm with whatever water rights it may have. If the use of this water for municipal purposes will injure other water users, outside of the town, then they should be recompensed. After a city or town buys water rights, they become preferred uses, so that the priority of the right makes but little difference. The main object is to see to it, at the time the transfer is made to the town or city, that all who might be injured by this change are taken care of at once. Section 726 provides that these changes shall take place under the direction of the Board of Control. This means that an inspection will be made by the Superintendent, all users will be notified and the final record will embrace a settlement that will be in harmony with physical conditions and with justice to every allied interest in the stream. One thing should be remembered—preferred uses are to be protected in seasons of scant water supply. When a change is made of any inferior right to a preferred use, all who might be affected should be made acquainted with the purpose of those who represent the preferred use and all precautions taken while the matter is before the board to secure whatever redress may be justifiable. It should also be remembered that preferred uses require small quantities of water compared with uses for irrigation. For instance, suppose a man owns 160 acres of land to which a water right attaches. Assume that he covers

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this to a depth of two feet during an irrigation season of 90 days. This is 320 acre-feet of water, or 104 544 000 gallons. This volume of water would furnish a water supply for a town of 3 000 people for a year, estimating that each person requires 100 gallons of water per day of twenty-four hours. Since the irrigator does not use water from the stream, except during the irrigation season of say 90 days, he has no right to the water during the remainder of the year. The town can file an application for the necessary supply outside of the irrigation season and the permit secured gives rights that cannot interfere with the rights of any other user, unless other rights have been secured which depend upon diversions outside of the summer season. The water right for 160 acres with the additional flow that could be secured under permit would supply a town of 12 000 people, if the water could all be used without loss. Certainly this should be a preferred use. The value of a city of 12 000 people to the State and to the public as locally represented, is far in excess of the value of any 160-acre farm, no matter how highly it may be cultivated.

"Another point should not be overlooked in this connection. The State does not permit the use of the legal maximum flow—one cubic foot per second for each seventy acres of land reclaimed, except when this volume can be beneficially applied. The owner of irrigated land cannot charge except for the value of the land with a water right, compared with the land in its former arid condition. He cannot, in other words, transfer to a municipality or to any preferred user, the maximum limit of his water right. This must be fixed by the board of control at such hearing as it may conduct. The city may buy the land outright and the entire tract of say 160 acres may be irrigated. The land may be of such character that one acre-foot per annum will raise crops and this volume may have been the maximum that has ever been used. Manifestly this is all the municipality obtains.

"It is believed by those who understand natural streams and the effect of diversions therefrom that it is much better to permit preferred users to purchase any rights that may be available rather than to place a premium on the first right, as has been the custom. For instance, if the first right were at the head of the stream and the city at or near its mouth, the irrigators above would lose all return seepage from the lands irrigated under the early right should this be transferred to the municipality. The early right and all other rights might continue undisturbed should it be possible for the town to secure a comparatively inferior right nearer to its boundaries.

"This legislation was not prepared on the spur of the moment. It represents the result of many years of study. It received the consideration of irrigation authorities throughout the West. It was given very careful study by the committees on lands and irrigation and by individual members of the Legislature of 1909. This act does not represent the personal views, only, of any individual or by any class of individuals. It represents in concrete form the wisdom of water users, students of irrigation and legislators. It passed the Legislature of 1909 with but seven dissenting votes altogether. No attempt was made to frame a measure of the kind until the views of the water users of Wyoming had been obtained. The responses received to letters mailed

to thousands of irrigators throughout the State laid a foundation for the bill as it was submitted. Mr. Johnston.

"It has been said that other States and that courts do not recognize the principles embodied in this act. It must be admitted that Wyoming stood for something of the kind long before any such doctrines were heard of outside of the State. It must be admitted that because other courts failed to get down to the proper fundamental principles, our courts had no guide, except in so far as our law-makers prepared the way. However, these doctrines are spreading. We need not search in vain for decisions that have the fundamental principles clearly defined. On March first, 1910, a decision was handed down in Arizona which should be read in full by every student of irrigation. The title of the case is 'Patrick T. Hurley, the United States, Intervenor, *vs.* 4 800 other water users.' It relates to the settlement of claims to use water from Salt River. The Reclamation Service is constructing one of its large projects in the valley of this stream, hence the United States intervened in the suit. The Supreme Judge of the Territory, acting as District Judge, heard the evidence and made the decision. It is remarkably complete. The definition of principles appearing on pages 8 and 9 of the printed copy of the decision is worthy of careful study and consideration. We quote this in full as follows:

"'The doctrine of riparian rights does not obtain in Arizona. The right of the owner of land to divert from a natural non-navigable stream the flow of water therein and to apply the same to beneficial use upon such land, is and always has been recognized in this territory. Such diversion and use is termed an appropriation of water. Whatever may be the steps necessary to take to initiate such a right or to evidence the intent to initiate it, the appropriation itself only becomes complete and vested when the water is actually diverted from the stream and placed to a beneficial use upon the land. The right given by such an appropriation is strictly not a right to the water itself, but a right to the use of the water. Its application to a beneficial use upon the land is as necessary in order to complete the right as is the diversion thereof from the stream. An appropriation of water, therefore, for the purpose of the irrigation of a parcel of land may not be established and completed by means merely of a declaration of intention or by the posting of notices of appropriation, nor may it be made by a canal owner or by a canal company as such alone, independent of its ownership of the land; but as application to a beneficial use upon the land is necessary to complete the appropriation, it follows that such appropriator must be an owner of land or have a possessory right thereto. Furthermore, since the land to which the water is to be applied is a necessary integral part of the appropriation and a factor by which the amount of the water appropriated for use is measured, it follows that when the water is no longer applied to the land for which it was diverted, the right of appropriation of such water for such land ceases. The right of appropriation further depends upon a supply of water that is unappropriated. It follows, therefore, that the first in time of appropriation is the first in right to appropriate, since water previously appropriated by another is no longer available for a subsequent appropriator. The extent of the appropriation is limited by the beneficial use to which the water can be applied. The actual amount

Mr. Johnston.

of water that may be appropriated for irrigation, therefore, is the amount that the land owner can and does actually use in the necessary and economical irrigation of his land for cultivation. This much and no more may he have; and this much he may only have when there is sufficient water available to supply first those prior in date of appropriation. The fundamental principle in the doctrine of appropriation of the normal flow of water in a stream for irrigation is its application by the land owner to the land for a beneficial use. The right to appropriate is a right that belongs to the land owner, but the water appropriated is appropriated for the land, and when so appropriated its use belongs to the land and not to the appropriator. The method of diversion from the river and the means of carriage of the water to the land is immaterial in the establishment or maintenance of the right; it may be done by the individual appropriator or by an association of individual appropriators, or by a canal company, or by any person or corporation; and the means of carriage or the point of diversion from the river may be changed from time to time to suit altered conditions without impairing the right of appropriation already made, provided prior rights of others are not interfered with. There being in this territory no private property in water, but water being a public property subject to the uses before defined, in so diverting and carrying the water such person, association or corporation acts merely as the agent of the appropriator and acquires no right of appropriation to the water itself, and no rights as against the appropriation made to the land, except a right to proper compensation for such diversion and carriage.'

"This decision marks distinct progress in court decisions relating to the use and diversion of water from natural streams. The principles upon which the decree is rendered afford protection to every claimant, yet when they are applied in practice no man is given a weapon whereby he may destroy the prosperity of his neighbors. The personal element is eliminated. The welfare of the community, through the protection of each individual in accordance with the nature and extent of beneficial use of water, is taken as the criterion for the settlement of rights rather than the claims of individuals regardless of the character of the irrigation development they may have been responsible for. Because the federal government intervened in this case and because the judges were at the time federal officers, the decision has a value that is more far-reaching than would ordinarily be the case. The entire decision is recommended to the consideration of those who are interested in the settlement of water right claims and controversies.

"It has been held by those who do not support the principle which unites water rights to the land reclaimed that the act of 1909 does not apply to water rights which were perfected prior to the passage of the law. This is the old argument that supports 'vested rights' regardless of the welfare of the public. The public and communities generally will protect individuals in their rights if the public and the community is made supreme over the individual. If the individual is superior he will take advantage of his position to injure the community. If the State cannot make laws affecting the administration of its own property, then it certainly cannot enact statutes which have for their purpose the collection of taxes, for instance, which relates to

the raising of revenue from property in private possession. It cannot quarantine live stock belonging to private parties even for the protection of communities. It cannot pay bounties for killing predatory wild animals or defray the cost of protecting game animals. To say that the State cannot regulate the use of water by the enactment of new legislation, even after water rights have been acquired for various purposes, is on par with the claims of certain people who held that because they began the use of water prior to the admission of Wyoming to statehood, the State had no control. This contention led certain claimants in Johnson County to refuse to submit proof before the Board of Control and to attempt at a later date to secure certain rights by litigation. The case was finally decided by the Supreme Court of the State, 9th Wyoming Report, 110. This decision is plain. It indicates that the State can regulate the use of water regardless of when the claim was initiated or under what laws water was first used. It is not presumed that the Legislature will injure water users by the passage of any general laws. It is not believed that it will ever be necessary to injure any water user to protect a community. It is essential to have water rights defined in such a way that the individual will never presume to have such rights as will enable him to even threaten the prosperity of a community. If no water user has rights in excess of those that attach to his lands as limited by beneficial use, the community need never fear any trouble. The danger in irrigation matters never begins with the community, or the State. It has its birth in the greed of one or two who, through the weakness of supervision in behalf of the public, are able to get what they are not entitled to, thereby enriching themselves at the expense of their neighbors. Mr. Johnston.

"The fundamental principles which provide equal protection to all and special privileges to none, are so simple and their number is so small, that it is surprising that they are not stated in every irrigation statute and in every text book dealing with the use of water. Yet we look almost in vain for any discussion of them. All that need be borne in mind is that the right to use water should be limited in accordance with the beneficial use made and the right should belong to that use rather than to the user. All other matters relating to the right are questions of fact that are easily obtained."

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—Every great hydraulic project calling for a large expenditure of money demands the employment of three distinct classes of Man, namely, a good promoter or business man, a good lawyer, and a good engineer. Each of these men attends to his own particular line of business, and, as a final result, the scheme is a success. This seems to be the experience everywhere. Mr. Le Conte.

Of late years, however, the universal knowledge demanded of the engineer has grown to such an extent that, in order to be thoroughly up to date, he has to be well nigh omniscient. The time now seems to be fast approaching when the engineer in charge of a large scheme will be compelled to acquire the necessary knowledge of all three of these classes of men.

Mr. Le Conte.

The author says that Court decisions are based more on the logic of judges than on the statutes. This calls to mind the lamentable fact that almost everywhere all the law that the general public gets, at best, is judge law, pure and simple.

The author calls on the Engineering Profession to come forward and take active lead in the movement for the enactment of better water laws. This is badly needed everywhere, and certainly engineers are, by experience, better fitted for the task than men of any other class.

The author says that many lawyers fear that the reform in water laws would spoil their business, which is true; but it also shows—a well-known fact—that members of the legal profession, as a rule, make their living by feeding on the misfortunes of the public. It is natural, therefore, that they should object to anything that cuts down the business of the Courts.

It is highly desirable that interstate or National water laws should be framed and enacted as soon as possible, inasmuch as all large projects are now halted and waiting for results. The new State water-right laws of Oregon certainly seem to be simple and effective, and, apparently, they fulfill all public requirements very satisfactorily. It now remains to enact similar laws to cover the needs of interstate streams.

It will be remembered that the City of New York contemplated getting a new water supply from the Housatonic River, which, to a large extent, is in the States of Massachusetts and Connecticut. After much wrangling in the Courts, the scheme was abandoned, and the city was forced to go to the Catskills, which project is now being developed on a grand scale.

The report of the Committee on Interstate Water Rights to the National Irrigation Congress, held at Sacramento, Cal., in 1907, seems to be highly commendable. The equitable suppression of the drastic law of riparian rights seems to be absolutely necessary for the public welfare.

Finally, the writer would respectfully suggest, in keeping with what he has stated before, that the Administrative Board, which will have complete jurisdiction of all matters relating to water, should be composed of good engineers, good business men, and good lawyers, all of whom are reputable citizens. The reason for this is that it is now practically impossible to find one class of men gifted with all these qualifications combined.

The members of such a Board, of course, would shoulder grave responsibilities, and their compensation should be ample, so as to enable them to give their time exclusively to the Board's business.

Mr. Dillman.

GEORGE L. DILLMAN, M. AM. SOC. C. E. (by letter).—This paper brings up a subject which is most pertinent and timely. If it results

in a solution, more good will have attended it than can be put in dollars and cents, and everybody concerned will benefit: the investing public, the constructing companies, the consumer who ultimately pays the bills—everybody but the professional litigant. Mr. Dillman.

Some years ago the writer was employed as engineer for one of two strong litigants on a water-right case in Southern California. The usual slow course in the Courts was followed for years; a final verdict seemed impossible for years to come. Somebody, at a "psychological moment," suggested compromise, and, as far as the legal adjudication of the case is concerned, it will never be settled. After the case was dropped, one of the attorneys was asked: "What can make a good water right in California?" The reply was: "Nobody has any right to such a thing who is not prepared to fight for it at any and all times. There is no such thing as a perfectly defensible water right."

Two strong corporations in Kern County agreed to disagree over the water rights of Kern River, a large stream with many diversions for irrigation purposes. Their case was in the Courts for several years, during which time the small claimants allied themselves with one side or the other. Many thousands of dollars were spent, and the case was withdrawn without a verdict. The principal litigants agreed to divide the whole waters of the river between themselves, and the small fry were shut out. The legality of the dispute is not settled, but both big companies together stand ready to make it interesting for all adverse claimants. They have acquired adverse claims by prescription now, therefore, there is no question as to their "rights."

In California there are thousands of doubtful rights, involving millions of dollars, which would be settled if it could be done with small or reasonable expense. The owners of new diversions, changes of use, or changes of point of diversion, are not safe in their investments until their rights are prescriptive. Only a few days ago, one of the leading lawyers of California stated that the best water right to be had was by prescription and continual use, that filing rights and riparian rights were so indefinite that their defense was always doubtful.

There will never be much conflict between uses for power and irrigation in California by reason of the physical conditions, provided the power water is returned to the stream after use. In a large majority of cases the power sites are above the irrigation diversions. Where considerable areas of irrigable land lie in valleys above power sites, their irrigation will generally result in the steadier flow of streams below. Where power necessitates storage to increase low-water volume, this storage will benefit the irrigation below, because the irrigation season is generally the time of low water. Therefore irrigation and power uses can be made mutually beneficial.

Mr. Dillman.

This same question is international in places. Recently, the papers announced that the Mexican Company had shut the water from California in Imperial Valley. Only a few years ago the United States and Canada came to an agreement over the respective quantities to be diverted at Niagara Falls.

While agreeing with the author on the desirability of definite laws, uniform if possible, respecting water rights, there are some suggestions in the paper which seem wrong.

A National Board, if formed, should certainly have its jurisdiction limited to strictly interstate waters. The difficulty of getting action by any Bureau in Washington hardly warrants the establishment of a new one.

If the Government would grant permits for power development, considering the companies as public service corporations, and regulate them as completely as they regulate the railroads, the farmer would get his power much cheaper than under Government ownership.

There is a crying need for something definite in water-right law. The experts cannot tell what the law is, and the Court decisions are very contradictory. The engineers of California will all be glad of something that is determinable. The State Commission seems to have made progress in Oregon. Perhaps others will elsewhere. We should welcome an interstate commission if its jurisdiction covered only strictly interstate questions; but, please deliver us from any more Government Bureaus. Those we come in contact with are pernicious.

Mr. Moore.

W. E. Moore, M. Am. Soc. C. E. (by letter).—Mr. Lewis has contributed a most valuable paper. The need of legislation along these lines has been felt for several years in all the Western States, and will be felt sooner or later in every State in the Union. It is not any exaggeration to state that there is not a single country on the globe which has a water law in accordance with present-day needs.

Oregon certainly has the best law governing water rights of which the writer has any knowledge. It is a long step in the right direction. With its general outline he agrees most heartily, but thinks it does not go far enough—does not cover as much ground as it should. In some respects it should be more specific, for example, in defining the duties of the engineer in granting a right on a stream before he becomes thoroughly familiar with that stream. It requires several years to learn all the characteristics of any stream, and the writer is opposed to any one granting a water right unless the water is there to fulfill the grant. It is true that, in case of a shortage, the Oregon law shuts off the latest appropriators, but these appropriators ought to know whether they will get water for a part of every year, and what part. As every one knows, the low-water flow of every stream varies from year to year, and such data should be available in

the office of every State Engineer in order that the water users of that stream will know on what to depend. Mr. Moore.

There is a growing belief that every State law should limit the water in every case to the least quantity necessary. This is desirable for various reasons; it admits of a wider use of the waters, and, consequently, more water users, and it tends to prevent waste which sooner or later becomes a menace to the public health.

In the writer's opinion, every water contract should be submitted to the State Engineer for approval. In this way many of the contentions between unscrupulous promoters and *bona fide* water users will be avoided. He would also make all plans for the construction of hydraulic works subject to the approval of the State Engineer. There has been a great deal of speculating on water rights granted by State Engineers in several of the Western States, when in reality the grants amounted to nothing. The time has come when such a thing should be impossible. When the State Engineer grants a water right for any purpose, investing capital should be able to rest assured that the water is there, and that the system or plant by which it is to be used will be constructed along right lines. This is just as essential for the water user as it is for investing capital, for it is the water user who ultimately pays the bills, and the better he is protected in his rights, the better will it be for the lasting prosperity of the community.

That Federal legislation is needed on interstate streams is certainly patent to every one. The problem cannot be handled equitably by the different States until the nature of the human race is radically changed. These waters can be apportioned properly between the interested States by the National Government, and that certainly ought to be done without delay, as the problem becomes more complicated from year to year and retards progress very materially.

The writer certainly does not agree with Mr. Lewis in advocating the construction and operation of power-plants by either the Federal or State Governments, unless such work can be entirely and absolutely eliminated from politics.

One of the most valuable features in the operation of the Oregon law is shown by the fact that during the three years it has been in force, it has clearly demonstrated that practically all the contentions over water rights can be satisfactorily settled outside of the Courts. Even the legal profession admits that legal practice has become entirely too complicated and cumbersome, thereby causing injurious and frequently fatal delay. It is obvious, therefore, that anything that will hasten the settlement of contentions and lessen their cost will be a benefit to the community. The writer has no desire to deprive the legal profession of anything that rightfully belongs to it, but it is just as reasonable to contend that the engineer could settle legal

Mr. Moore. questions as it is to argue that the lawyer could settle engineering questions. It is a fact that many engineering questions are intimately connected with legal questions and cannot be separated from them, but as far as possible they should be kept apart. To the writer's mind this is one of the most important problems with which the Engineering Profession has to deal at the present time, and he feels very grateful to Oregon for the many valuable lessons it has given us.

Mr. Bien. MORRIS BIEN, ESQ.* (by letter).—This paper is very valuable in summarizing the present situation regarding water rights in the Western States, and will enable both engineers and lawyers to grasp more fully the great importance of the problems which are yet to be solved in regard to the determination of rights to the use of water.

In a country where most of the land is practically valueless unless it can receive a water supply for irrigation, it seems strange that it should take so long to provide for a satisfactory and conclusive record of water titles comparable with the record of land titles. The first step in this direction was taken by the State of Wyoming in 1890, and some of the other arid land States have followed along the general lines adopted there.

Large investments, running well into the millions, in connection with irrigation and water-power construction in the arid regions, depend fully as much on the title to the use of water required as on the title to the land whereon the structures are built.

Many enterprises of considerable importance have found themselves compelled to defend in the Courts their right to the use of water essential to the enterprise, the usual examination of the record in such cases giving practically no indication regarding the true condition of the water supply.

So long as it is possible in a number of the arid States for any one to place on record a claim to a quantity of water to be diverted from a particular stream far in excess of its flood flow, and entirely regardless of the fact that all or a large part of the available water supply may have been put to beneficial use, just so long will diastrous failures of water supply be encountered.

Even the Courts of some of the Western States have so far placed faith in such records as to issue decrees declaring water rights vested in litigants which could call for a water supply of many times the greatest known discharge.

The modern system of water law designates the State Engineer as an expert witness to determine all the features necessary for fixing the extent of the vested water rights and the quantity of unappropriated water which may be available for future enterprises. His findings are conclusive, if not appealed to the Courts, and, when thus

* Supervising Engineer, United States Reclamation Service.

appealed, his expert determination of the essential facts would not be disturbed except on proof of error. Mr. Bien.

This is radically different from the bewildering and contradictory statements by ill-qualified witnesses regarding flow of water, capacity of ditches, areas irrigated, etc., which even now are characteristic of litigation regarding water rights in a number of the Western States.

The system which is now being worked out in Oregon and in some of the other Western States will ultimately provide a record of titles to the use of water as reliable within the necessary limitations as our records of title to land under the most up-to-date systems, substantially equivalent to the Torrens system of land title records. The right to the use of water differs from a fee simple title to land in two essential particulars: first, there is no ownership of the *corpus* of the water as there is of the land, the right in the former case is only a right of use; second, the water of the streams is not fixed, but transitory, and, moreover, the quantity available fluctuates from day to day and year to year. Nevertheless, it is possible to provide for an adjustment of the respective rights to the use of the water, and to make such rights as easily determinable as the ownership of land.

The questions of transfer of the place of use and changes in the method of diversion are of extreme difficulty, and before transfer or changes are permitted without loss of priority careful expert investigation is essential.

In one case a considerable area of land bordering on a small stream with the appurtenant water rights was purchased and the place of use and method of diversion were radically changed. Instead of small ditches diverting the water short distances from the stream and after irrigation allowing return seepage to the stream, the entire quantity of water thus claimed was diverted in an iron pipe and carried to another water-shed. It is claimed by water users below this point of diversion that they have suffered a serious diminution in the quantity of water which had formerly been used on the land above and returned to the stream to become available for their use. They claim that the purchaser of the upper lands could not take a large proportion of the water out of the drainage area without seriously impairing their rights. It will doubtless require a decision of the Courts to settle this question.

The determination of water rights on interstate streams is one which, at an early date, should receive the attention of the Federal and State Governments. The plan suggested by Mr. Lewis is undoubtedly the only logical solution of the problem, and these two Governments must co-operate in working out its details.

The principle that water rights on interstate streams must be adjudicated independently of State lines, while indicated in a general way by the decisions of the United States Supreme Court, is not wholly settled.

Mr. Bien. In the case of Kansas *vs.* Colorado, the Supreme Court took the first step, but without definitely announcing such a principle. In the case of Bean *vs.* Morris (221 U. S., 485), the Supreme Court assumed that the States would recognize prior vested rights in another State affecting a stream common to both States.

The question, however, is not decided definitely by these cases, and it may be that the United States Supreme Court will lay down this rule definitely in the case recently begun in that Court by the State of Wyoming against the State of Colorado which seems to rest almost wholly on this question.

Mr. Sheley. HORACE W. SHELEY, ASSOC. M. AM. SOC. C. E. (by letter).—For the sake of clearness, the writer has divided his discussion of this timely paper into five parts. Although he has made special mention of irrigation rights, he believes, with Mr. Lewis, that the same principles can be applied to the use of water for other purposes.

1.—Measurement of Water.—Before it will be possible to make water rights definite, certain modifications in the units of measurement must be made. Most of the Western States have already abandoned the variable "miner's inch" for the cubic foot of water per second, commonly called the second-foot, but this is a rate of flow and not a quantity; this term conveys no more meaning than the answer "sixty miles an hour" would give to a question about the distance by railroad between two cities. So far as the writer knows, Nevada is the only State which has named a quantity of water, instead of a rate, when fixing water rights. In that State the unit of measure is the acre-foot. Some other States mention the acre-foot in the regulations of their State Engineer's Offices, but do not give it in their statutes.

A further step remains, namely, to fix the maximum rate at which a given quantity of water may be taken; this corresponds to the "peak load" of electrical engineers.

2.—Place of Measurement.—For new irrigation enterprises it is generally better to name some point near the place of use as the point of measurement, in retailing water, because then the wholesaler, or constructor of the irrigation works, cannot be accused of not having properly constructed his reservoirs and canals so as to prevent seepage and waste in transit. If this method had been followed by a certain large company in one of the arid States, it would not now be in legal difficulties over the failure of the floor of a reservoir to hold water, because it could deliver the requisite quantity from another reservoir.

Measurement at the place of use makes it possible for a later comer to improve the ditch of the old appropriator, and take the water formerly wasted in transit for use on his own land. It is an incentive to the older appropriator to make these betterments himself, lest some one else do it and take this water from him.

Mr. Sheley.

3.—Changes in Place or Manner of Use.—Because of his experience, both in Utah, where changes in the place and manner of use are permissible, and in adjoining States, where the water is appurtenant to the land, the writer believes that the Utah policy is the better, leading to higher duties and more economical use of the water.

If a farmer has more water than he needs for a given tract, as a result of an excessive appropriation in the beginning, of a change in crops, of more thorough cultivation, or of a rise in ground-water, he will continue to use all the water if he cannot transfer all or part of it to other land without loss of priority. If the law permits the transfer under proper safeguards to others, it leads to greater duty for the water.

On the other hand, if the water is appurtenant to the land, so that the owner cannot transfer any part of it, it will be necessary for any one desiring to use it to go to the Courts or to a special tribunal, in order to do so, and the difficult burden of proving that the present owner is not applying the water to the best advantage will be on the new comer.

It is sometimes desirable *pro bono publico* to change the manner of use. The laws of Wyoming recognize that some uses are more important than others, and state the preferences, as follows:

"First—Water for drinking purposes for both man and beast. Second—Water for municipal purposes. Third—Water for the use of steam engines and for general railway use. Fourth—Water for culinary, laundry, bathing, refrigerating (including the manufacture of ice), and for steam and hot-water heating plants. The use of water for irrigation shall be superior and preferred to any use where turbines or impulse water-wheels are installed for power purposes."

4.—Adjudication of Water Rights.—In 1908 the writer assisted the Hon. Caleb Tanner, State Engineer of Utah, in the tabulation of the statements of 1 200 or 1 300 claimants of water rights in the Weber River water-shed, and since then he has had small respect for the truthfulness and reliability of the claims of owners of water rights. Most of the claims were exorbitant. Our Courts have not been averse to granting absurd claims, and in one instance a judge decreed more than 14 ft. depth as the necessary quantity for a tract of land, where 3 ft. would have been ample.

The system of adjudication in Utah has failed completely, thus far, through lack of money to complete the adjudications and the inherent defects in the method used there. The surveys for the adjudication of the Weber River system were started in 1903 or 1904, but the matter has not even reached the District Courts.

The writer believes that the system first used in Wyoming and later modified and used in Oregon, is the only one that is practical.

Mr. Sheley.

5.—Interstate Rights.—The position taken by Mr. Morris Bien in the quotation given by Mr. Lewis, to the effect that the States have not now, and never have had, a right to control the waters within their boundaries, seems to be extraordinary, in view of the fact that the United States Government permitted the Territorial Legislatures to make water laws, and it has applied to the States for water rights, as if it were a private corporation, when undertaking irrigation enterprises under the Reclamation Act. Possibly the context of Mr. Bien's statements would explain the paragraph quoted.

All who have had any experience with the red tape and delays at Washington will hesitate about surrendering any State rights to it, unless absolutely necessary.

In conclusion, the writer agrees with others who are studying water laws and the use of water, believing that the principles of preferential use, beneficial use, and priority can and will ultimately be adopted in the determination of rights both in one State and between States.

Mr. Knowles.

MORRIS KNOWLES, M. AM. SOC. C. E.—The speaker is particularly grateful to the author for his clear exposition of the doctrine of appropriation as practiced in Oregon; and is personally interested in the discussion of this important question, as he has the honor of being President of an Association, one important object of which is the improvement and rationalization of the water laws of Pennsylvania. The paramount importance, at this time, of flood prevention and protection, power development, water supply, improvement of navigable rivers, and the whole broad subject of water conservation in every part of the Union, points to the necessity of adopting in every State a well-considered, comprehensive plan, based on adequate State and National legislation.

The speaker agrees most heartily with the suggestion that the Engineering Profession generally, and engineering societies particularly, not only can, but ought to lead in the discussion of every phase of the subject, and lend their aid to the framing of legislation on such a matter, for the consideration of which their members are peculiarly fitted, in many respects better than any other class of citizens. It is time we were putting into action in this field the policy urged by President Ockerson at the Seattle Convention.

Prior Appropriation vs. Riparian Law.—On one point, however, the speaker holds an opposite opinion from the author, which illustrates the peculiar situations that may arise when engineers undertake the discussion of the intricacies of the law.

The case of Kansas *vs.* Colorado did not, in the speaker's opinion, declare that "the doctrine of riparian rights, * * * is not the law, and therefore never has been the law," even of interstate waters. On the contrary, Kansas *vs.* Colorado definitely followed the principles of

Mr. Knowles.

riparian rights, as between Kansas and Colorado. The fact that the Colorado use in question was on non-riparian lands did not affect the issue. The Court considered the decision from the point of view of the rights of Kansas as a State against those of Colorado as a State, and not of the rights of individual riparian proprietors in Colorado. These individual rights come under the rule previously enunciated in Anderson *vs.* Bassman (140 Fed., 22) and later affirmed by the Supreme Court in Rickey, etc., Co. *vs.* Miller (218 U. S., 258) that the riparian owner deduces his rights from the law of his own State and that the private right is "not in his own right, but by reason of and subordinate to the rights of his State" (Turley *vs.* Farman, 114 Pac., 278). In the case of Rickey, etc., Co. *vs.* Miller, which involved the rights of riparian owners in California and appropriators in Nevada, the rule followed was that

"the enforcement of either right beyond the boundary of its State must depend upon the concurrence of the other State. Unless the upper State (California) will voluntarily impose conditions upon its citizens in favor of users in the lower State (Nevada) the latter have no right in the matter other than to complain that the lower State as such (and not merely the plaintiff) is not receiving an equitable share of the benefit of the stream."*

The language of the Court in Kansas *vs.* Colorado makes plain that the principle of equal rights, which is the basis of riparian rights, was followed. For example, referring to the law in Kansas, which is the law of riparian rights, Justice Brewer said:

"As Kansas thus recognizes the right of appropriating the water of a stream for the purpose of irrigation, subject to the condition of an equitable division between the riparian proprietors, she can not complain if the same rule is administered as between herself and a sister State."

(The use of the word "appropriating" here appears to the speaker to be somewhat unfortunate, but need cause no confusion, for it bears no connection whatever with the doctrine of "appropriation" as practiced in other Western States. Apparently, the word "diverting" would have expressed the idea of the learned judge without danger of confusion.) And again, referring to the statement quoted by the author, from the conclusion of the Court, in the Kansas *vs.* Colorado case,

"* * * it is obvious that if the depletion of the waters of the rivers by Colorado continues to increase, there will come a time when Kansas may justly say that there is no longer an equitable division of benefits, and may rightfully call for relief against the action of Colorado."

No support is given by this language to the theory that there may exist here any exclusive right by priority. This is the language of the riparian rights doctrine.

* Wiel, "Water Rights in the Western States," 3d ed., p. 364.

Mr. Knowles.

Then, since the premise is in error, the conclusions that "riparian rights are not considered as vested rights by the Supreme Court," and that "if Congress should decide that the enforcement of the doctrine of priority of appropriation and beneficial use, * * * should constitute an equitable apportionment of benefits, it is conceivable that the Supreme Court would uphold such act," must fall. In the speaker's opinion, such an act, in the light of the present generally accepted views of the powers of Congress, would be unconstitutional, both as outside the enumerated powers of the Federal Government, and as an invasion of the right of private property. In fact, the Kansas *vs.* Colorado decision says:

"It [each State] may determine for itself whether the common law rule in respect to riparian rights, or that doctrine which obtains in the arid regions of the West of the appropriation of water for the purposes of irrigation shall control. Congress cannot enforce either rule upon any State."

We of the East and the country at large must look to other measures than the adoption of the law of appropriation in its entirety for the development and conservation of our water resources.

Executive Board vs. Court Administration.—On the other hand, the speaker agrees with the author in believing that water laws may be better administered by executive boards than by the Courts and by injunction; and he believes that it might be not only lawful, but very serviceable, for Congress to provide the machinery for administering the law with respect to interstate waters, provided the law can be said to be already determined by Court decision. This, however, does not give ground for the broad assertion that "Congress must have power to prescribe by law what shall constitute an equitable division of benefits as to interstate waters." In fact, as already stated, it would seem plain from Kansas *vs.* Colorado that Congress does not have that power—a very different one from the power to provide the machinery for administering the law after the decisions of the Supreme Court have prescribed what shall constitute an equitable division of benefits.

This does not in any sense mean that the speaker does not believe that the Federal Government, under the Constitution, can co-operate with the States in exercising its control over the navigability of streams, and their tributaries also, in such a way as to secure great collateral benefits. On the contrary, he holds that opinion most strongly, and believes in the propriety of the adoption by Congress of the Newlands River Regulation Bill and similar measures.

There may be a question, also, as to whether the law has become sufficiently determined to justify the creation of an administrative commission. The doctrine in Kansas *vs.* Colorado was clearly stated; but whether this would apply under all conditions is rendered some-

what doubtful by the following from the decision in Rickey, etc., Co. *v.* Miller (218 U. S., 258, 261). Mr. Knowles.

"It is conceivable, to be sure, that the decisions of this Court may determine that the States have rights as against each other *in invitum* in streams that flow through the land of both. (Kansas *v.* Colorado, 206 U. S., 46, 84; Mo. *v.* Ill., 200 U. S., 496, 519, 520.) The rights may vary according to the system of law required by natural conditions. They may be more or less analogous to common law rights between upper and lower proprietors, where irrigation is not necessary, as in most of the older States. (See N. Y. *v.* Pine, 185 U. S., 93, 96.) There may be some, perhaps limited, right of appropriation in the upper State, at least in the water-shed of the stream, where irrigation is the condition of using the land. (See Kas. *v.* Col., 206 U. S., 46, 100-104, 117.) But whatever this Court may decide, if a private owner should derive advantage from such a decision, it would not be in his own right, but by reason of and subordinate to the rights of his State. * * *"

If it be true that the law for all cases has not been determined, it would appear almost certain that any important contest between States, before an administrative board, would be appealed to the Supreme Court, until the universal law had become established, and a Commission might therefore be of no value at present. In addition, it is debatable whether enough important contests between States will arise to require the continuous service of a Commission.

State Commission Administration.—None of these objections applies, however, to administration of intra-state water laws, on a basis of equitable apportionment, by State Commissions. States undoubtedly have power to determine by law what shall constitute an equitable apportionment of the use of water within their borders; and the delegation of this power to a commission cannot be opposed on the ground that it must be exercised only by direct action of the Legislature, any more than the regulation of public utilities by Commission, now so firmly established, can be so attacked. The words of Justice Timilin, in Minneapolis, etc., Railway Co. *vs.* Wisconsin Railroad Commission, are applicable:

"It is argued that the power to fix rates is a legislative one and can never be anything else; * * * that the legislative power is by the Constitution vested in the Senate and Assembly, and cannot be set apart except as expressly provided for in the Constitution; but when we add to this that, because of the multitude of detail, the intricacy of the subject, the expert knowledge required, the numerous separate investigations of inter-related questions of fact which are necessary * * * a legislative body * * * would find it an actual rather than a legal impossibility to fix just and reasonable rates, it becomes apparent that this position tends to the conclusion that the State * * * was shorn of some of its usual and necessary power of sovereignty and became impotent to exercise the power of regula-

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tion. Regulation by direct action of the legislature has been tried and found impracticable and its attempt generally abandoned."

The speaker has not yet attempted to work out the details of such an administrative riparian system, but believes that it may be possible, by making simple and rapid the determination of riparian rights and of the terms of an equitable division by a suitably constituted commission, and by making proper provision for the condemnation of such rights for public uses under the supervision of such a board, to develop a system which will lead to a full utilization of the waters of a State, with ample protection both to the public and the investor, and with a possibility of obtaining in some instances great collateral public benefits in the way of flood protection, development of water power, improvement of navigation, improvement of quality of water, etc.

The feasibility of such a scheme appealed to Wiel, who says:*

"This system of law would seem to offer a field for administrative legislation; in fact, a readier field than the law of prior appropriation. Where the test is what is reasonable in each case, discretion must necessarily come into play, whereas where parties have exclusive rights measured by priority there is * * * little room for the exercise of discretion by administrative officers * * *. Where the common law applies the test of reasonableness, legislation is apt and readily applied; as, for example, in dealing with public service companies. The common law says their rates and regulations must be 'reasonable,' and accordingly public service commissions and similar bodies are created. Likewise under the new law of percolating water 'reasonable use' has become the test, and statutory regulation based thereon is being adopted. As yet, however, there has been no attempt to provide a statutory system governing the reasonable use of water by riparian proprietors among themselves, in jurisdictions applying that system, though there would seem a clear field for such legislation if desired."

Limit of Appropriation by Reasonable Use.—The speaker would like to call attention also to the unmistakable tendency (illustrated by the "Pro-rating" statute of Washington, the constitutional limitation of the right to appropriate in Idaho, and such decisions as Basey *vs.* Gallagher, 87 U. S., 670; Union Mining Co. *vs.* Dangberg, 81 Fed., 73; Anderson *vs.* Bassman, 140 Fed., 14; and Schodde *vs.* Twin Falls L. & W. Co., 161 Fed., 43) to depart from the strict law of prior appropriation, and to limit appropriation by a requirement of reasonableness, which is doing much to narrow the gulf between the doctrines of appropriation and riparian rights. It appears to him that, as density of population increases in the Far West, this tendency will increase; and that uniformity in the administration of State laws will be approached, in spite of differences in form, by the application of the "rule of reason" to both systems of law, rather than by the abandonment of either one in favor of the other.

* "Water Rights in the Western States," 3d ed., p. 830.

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On this phase of the subject, Mr. Morris Bien, Supervising Engineer of the U. S. Reclamation Service, speaking before the National Irrigation Congress at Spokane, in 1909, said:

"The doctrine of rights by prior appropriation has been adopted in nearly all the States where irrigation is required; but this doctrine as now generally understood will necessarily require modification. * * *

"In the Yale Law Journal for January, 1909, is a discussion of the idea of reasonable use, whether under the doctrine of riparian rights or the doctrine of appropriation. It shows that the courts have frequently called attention to the fact that the doctrine of appropriation must be modified by the idea of reasonable use which is also a fundamental limitation of the riparian doctrine. This idea of reasonable use will undoubtedly become an important factor in future years when valuable interests depending upon the entire water supply have grown up within many of the irrigation districts, and it becomes necessary to protect these interests in cases of temporary deficiencies which sometimes continue for a number of years in succession. * * * The qualification of the doctrine of prior appropriation by the idea of reasonable use, and the application of the same idea to the riparian doctrine will undoubtedly bring these opposing doctrines much closer together in actual practice, and is likely in the end to cause a practical uniformity in the governing principles of all the irrigation States."

Best Development of Water Resources.—In conclusion, the speaker wishes to raise the question whether the "Water Power Policy" advocated by the author is necessary to attain the desired ends. If so, the speaker would agree that "the States, in co-operation with the United States," should "develop this power and supply at cost plus interest." But, if it is possible to attain the same ends in other ways and without the tedious delays that must precede such a consummation, the speaker does not see the necessity of adding further commercial enterprises to the burdens of our State Governments. In the belief that, under a rational, well-defined system of water laws, and with wise State regulation in the interest of the people, private capital would construct the works necessary to the conservation of water, the Pennsylvania Water Conservation Association has been formed. This Association, including in its membership capitalists, publicists, civic bodies, power companies, and water companies, has for its object the formulation of a plan for the development and utilization of the water resources of the State, by means of private capital, under improved laws and State supervision, in such a way as to offer a safe, attractive field for investment; to insure reasonableness of rates and safety of construction; to secure, wherever possible, by supervision of designs and operation, prevention of floods and improvement of rivers; and to serve by a broad, far-sighted policy of conservation the public interests of this and of future generations.

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KENNETH C. GRANT, ASSOC. M. AM. SOC. C. E.* – The speaker is particularly interested in this valuable paper because of the direct applicability of the fundamental principles therein stated to the broad policy of water conservation which is now being advocated so strongly in Pennsylvania by the Flood Commission of Pittsburgh and the recently organized Water Utilization Association of Pennsylvania.

The Flood Commission was organized by the Chamber of Commerce of Pittsburgh, in 1908, to investigate the causes of, and damage by, the frequent disastrous floods at Pittsburgh, and to study and decide on methods of relief. After four years of exhaustive investigation, for which about $125 000 (obtained from county and city appropriations and by private subscriptions) was expended, the report of the Commission, published in April, 1912, recommended the construction of storage reservoirs on certain tributaries of the Allegheny and Monongahela Rivers. The flood relief which would be obtained by these reservoirs, naturally, would not be confined to Pittsburgh, but would extend to many other communities on the rivers above and below that city. Moreover, the proposed reservoirs would not only provide this flood relief, but would cause an increase in low-water flow which would add greatly to the usefulness of the streams for navigation, water supply, and power purposes.

The Commission pointed out, therefore, that the solution of the problem was so broad in its scope, and so far-reaching in its benefits, that it did not concern the City of Pittsburgh alone, but demanded State and National consideration and co-operation. The decisions to which these investigations have led have been aptly stated by the author when he says:

"Some interstate legislation is absolutely essential if the highest development of our streams is to be accomplished. Each river, from its head-waters to its mouth, should be treated as a unit, regardless of State lines."

The Water Utilization Association of Pennsylvania was formed for the purpose of framing and obtaining legislation which should bring about the fullest development of the water resources of the State, and at the same time preserve and dedicate the benefits of such development to all the citizens of the Commonwealth.

This Association and the Flood Commission have given careful consideration to the same thought as that embodied in the author's statement:

"Soon the State and Nation must join in the storage of water for the control of floods and to aid navigation. This water, in passing down the stream, will benefit many private power projects, and these should be compelled to contribute to the cost in proportion to the benefits received."

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* Now M. Am. Soc. C. E.

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These bodies, in their work, have also in mind the natural converse of this statement, that private water-power companies, proposing to construct large storage works which will assist in providing flood relief and will improve the rivers for navigation, water supply, and water-power purposes, should receive corresponding co-operation and assistance from the State and National Governments. There are instances in Europe where private water-power companies have been assisted in the construction of their storage reservoirs by the Government in return for a certain amount of additional storage capacity to be constructed and kept empty for flood control. Private users of water for domestic or industrial supply, or for power, located below such large storage works, should also contribute to the cost of their construction in proportion to the benefits received. If effective legislation providing for such co-operation can be framed and enacted, it will ensure the fullest development of the water resources of the State.

Legislation tending toward this end has in fact already been passed in Pennsylvania. At the last session of the Legislature, a bill, drafted by the Flood Commission, was introduced and passed, enabling counties to borrow and expend moneys for the construction of works for flood relief, and also to enter into contracts with each other, or with the State, with other States, or with the United States, for the purpose of carrying out the necessary works.

It is hoped that legislation broadening the powers of the Water Supply Commission of Pennsylvania may be obtained at the coming session of the Legislature. This Commission, in operation since 1905, is charged with obtaining such complete knowledge of the water resources of the State as shall enable it to provide for their most equitable distribution. It has power over the granting of charters for water-supply and water-power companies, but has no authority where the water is taken by corporations for their own use, by private individuals, or by municipalities. Its effectiveness, in carrying out the purpose named in the act creating it, therefore, is considerably restricted. Its jurisdiction should be widened to cover all users of water.

Within the limits of its powers, the Water Supply Commission of Pennsylvania has done most admirable work, some of it along lines similar to that of the Board of Control described by the author; for example, the work of the Oregon Board for the protection of the public interest, of which the author says: "Public interest demands that water be put to the highest use. * * * an application for either irrigation or power can be denied if it is in conflict with the higher use for domestic supplies," is identical in principle with the attitude of the Pennsylvania Commission in considering applications for charters. There are instances in Pennsylvania where prior applications for charters for water-power companies have been refused on

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the ground that the stream involved was needed for domestic supply. There are also cases where applications for charters for water companies for domestic supply have been refused, because they interfered, perhaps purposely, and certainly needlessly, with proposed water-power projects.

Charters for water-power companies have also been approved, with certain conditions. For instance, the charters for a large water-power project in Western Pennsylvania were approved recently, on certain conditions, notable among which were the protection of the interests of navigation and of riparian owners below the dam by the maintenance of a suitable minimum stream flow, the presentation, within 12 months, of data showing the extent to which the proposed reservoirs can be used to ameliorate floods, and the placing of the operation of the reservoirs, in so far as the control of floods and the maintenance of low-water flow is concerned, under the direction and jurisdiction of the Water Supply Commission. Had a similar commission been in existence in West Virginia, the plans of a large water-power project, recently chartered and about to begin actual construction, might readily have been enlarged and modified, to the great advantage of both the water-power company and the general public.

A thorough knowledge of stream flow throughout the State is also being acquired by the Water Supply Commission, through the operation of a large number of gauging stations. A complete collection of all existing stream-flow data, some of the records extending back for many years, is now being compiled, and will shortly be published. The Commission has also collected and filed complete statistics of all water and water-power companies in the State, including municipal water supplies. These data are also briefly shown on large-scale county maps of the State, which are convenient for reference in studying the relation of proposed to existing uses of water in a given region. It is evident, therefore, that in Pennsylvania, there is already in existence a body which, if given larger powers, can effectively protect the public interest and assure the most equitable distribution of the waters of the State.

Referring again to the treatment of a stream as a unit from its source to its mouth, there are several interesting and instructive examples of this broad policy in Europe, which the speaker has had the opportunity to examine on two occasions during the last few years.

The largest stream thus treated is the Ruhr River, which empties into the Rhine on the right bank at Ruhrort, in Western Germany. This stream has a length of 143 miles from source to mouth, and drains an area of 2 041 sq. miles. It flows through the great industrial region of Germany, around Essen and Mülheim, and is extensively used as a source of domestic and industrial supply and for power. The demands on the stream became so great that during low water

the supply threatened to be inadequate. In 1897, after long deliberation and much difficulty, a voluntary association was formed by the users of the Ruhr water for the purpose of improving the flow of the river. The membership of this association includes cities, factories, water-power companies, etc. There are eleven dams on the tributaries of the Ruhr, ten of which were built by smaller associations of the water users on the respective tributaries, and one, now nearing completion, by the Ruhr association. These smaller associations were formed under a Prussian law. After two-thirds of the water users below the proposed dam have agreed to form such an association, the other third is obliged to join with them. The main association of the Ruhr, called the *Ruhrtalsperrenverein,* was formed voluntarily, without such a law; but it can compel users of water from the Ruhr to pay into the association, in the case of water-power plants, according to the head and the quantity of water used, and, where water is taken for domestic or industrial supply, according to the quantity used. The assessments on the members of the smaller associations on the tributaries are made in a similar manner. The main association of the Ruhr pays about $70 per annum per 1 000 000 cu. ft. of storage capacity of the reservoirs to each of the ten associations on the tributaries. Mr. Grant.

Another example of similar co-operation in river regulation is to be found on the Wupper River, the next tributary of any size emptying into the Rhine on the right bank above the Ruhr. It has a drainage area of about 240 sq. miles, and flows through the thickly populated manufacturing region around Barmen and Elberfeld. Before the construction of reservoirs on its tributaries, the stream had a very irregular discharge, varying between 0.05 and 90 sec-ft. per sq. mile of drainage area. Flood damage was frequent and considerable, and the numerous water-power plants suffered greatly from low water.

After long deliberation, an association for the construction of reservoirs in the drainage area of the Wupper was formed under a special Prussian act. This association has built three reservoirs on tributaries of the Wupper, for the control of floods and the increase of low-water flow. All water users below these dams must pay an annual assessment to the association.

A third association, of similar character, has constructed six reservoirs in the Görlitzer Neisse, near Reichenberg, in Bohemia. This stream rises in Northern Bohemia and flows northward for 124 miles, emptying into the Oder River about 15 miles below Crossen, in Prussia. Its valley receives a heavy precipitation in its upper portion (13.5 in. in 24 hours having been recorded at some points) and has been repeatedly devastated by floods; therefore, after the great flood of 1888, an association was formed to plan and carry out the con-

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struction of protection and regulation works, consisting of widening and straightening the channel and raising and protecting the banks. The estimated cost of these improvements was so heavy, and their probable effectiveness in a great flood so doubtful, however, that practically nothing was done by the association, the actual work confining itself to repairing damages and building the bank protections most urgently needed by the individual property owners.

In July, 1897, this part of Europe was visited again by devastating floods, which revived public interest in flood relief to such an extent that a convention, in which all the neighboring cities and towns were represented, was held in Reichenberg in the fall of that year. At this meeting it was decided to investigate the feasibility of constructing reservoirs for flood control. In January, 1901, the preliminary studies were sufficiently complete to establish the general plans, which contemplated the construction of six reservoirs in the neighborhood of Reichenberg, controlling the run-off from about 29 sq. miles, and in critical flood time holding back about 3 530 sec-ft. of damaging flood discharge.

The result of the investigations gained the association many new supporters, and assured the sympathy of the population of the entire valley of the Neisse with the project. In fact, one of the most noteworthy features of this undertaking is the widespread interest it aroused in the surrounding country and the universal financial support it received. Although all the reservoirs are located in Bohemia, the benefits, both in flood control and increase of low-water flow, are felt by the Saxon and Prussian interests along its lower course, and these two countries, together with various cities, communities, and private interests, co-operated with Bohemia in their construction. The total cost of the work was $1 320 000, and the following contributions show the extent of the co-operation:

Bohemian Government	$660 000
Prussian Government	38 400
Prussian Province of Silesia	9 600
Prussian County of Ober-Lausitz	14 400
City of Görlitz (Prussia)	14 400
Saxon Government	24 000
Combination of Saxon and Prussian water interests	72 000

It is also of special interest that the users of water for power development from three of the reservoirs pay $12 per h.p., and from the other three reservoirs, $28 per h.p. per year. Three of the dams for these German reservoirs are shown by Figs. 3, 4, and 5.

As an instance of what such co-operation might accomplish in the United States, the conditions in the Beaver Valley in Western

FIG. 3.—DAM BUILT BY ASSOCIATION FOR IMPROVING LOW-WATER FLOW OF RUHR RIVER, GERMANY.

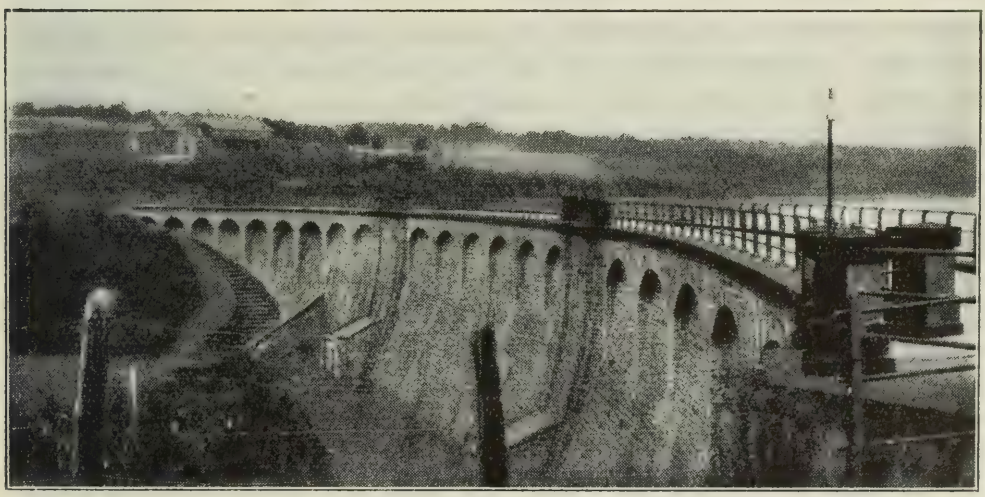

FIG. 4.—DAM BUILT BY ASSOCIATION FOR CONTROLLING FLOODS AND INCREASING LOW-WATER FLOW OF WUPPER RIVER, GERMANY.

FIG. 5.—DAM BUILT BY ASSOCIATION FOR CONTROLLING FLOODS AND INCREASING LOW-WATER FLOW AT REICHENBERG, BOHEMIA.

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Pennsylvania may be mentioned. This stream drains about 3 040 sq. miles, and empties into the Ohio from the north about 25 miles below Pittsburgh. It is extensively used as a source of domestic and industrial supply, as it flows through a thickly-populated manufacturing district. It is also used considerably for power. On its upper waters there is an ideal reservoir site where a low earthen dam would create a reservoir of large capacity, overflowing a large tract of useless swamp land. This site has already been studied as the source of feed-water for the proposed Lake Erie and Ohio River Canal. It has also just been carefully surveyed and mapped by the State Water Supply Commission, under a special appropriation by the Legislature, for the purpose of determining the possibilities and best use of the site. Here is an ideal condition for a *Beavertalsperrenverein,* which could be made up of the water users in the Beaver Valley and the Canal Company, with the possible addition of the communities along French Creek, a large northern tributary of the Allegheny, because of the flood control that would be afforded by a second reservoir proposed by the Canal Company on the north branch of this creek, which would impound the flood-waters of that stream and deliver them through a canal into the reservoir on the Beaver River head-waters.

Another example of what might be done by such co-ordination of interests is afforded by the Youghiogheny River, a stream rising in Northern West Virginia, and entering the Monongahela 15 miles above Pittsburgh. This stream is used extensively as a source of industrial and domestic supply, and has several favorable sites for water-power development. The Flood Commission of Pittsburgh proposes to store part of its flood run-off for the control of floods and for the improvement of the low-water flow, this stream being one of the chief offenders in Pittsburgh floods. The United States Government proposes to slack-water the lower 19 miles, to provide navigation up to West Newton, although a study of the low-water flow indicates that a pool-full stage could not be maintained without the assistance of additional water from storage reservoirs during dry weather. The co-operation of all these interests in the construction of storage reservoirs for the regulation of the flow of this river would unquestionably be of great benefit to all concerned. It would make feasible the development of a large quantity of water power which cannot be developed economically if the entire cost of the storage works must be borne by the water-power interests.

A third example may be found in Eastern Pennsylvania, where a large water-power project in operation on the lower Susquehanna River could greatly increase its capacity if the low-water flow were increased by storage reservoirs on the upper waters. The value of such increased flow to this one power plant would not pay for the cost of constructing such reservoirs; but if other plants, made feasible

Mr. Grant. by the increased low-water flow, were constructed, and contributed to the cost of the storage works, and if co-operation were also obtained from the communities, counties, railroads, and other interests damaged by floods, such a broad treatment of the river would undoubtedly become feasible.

Mr. Hopson. E. G. Hopson, M. Am. Soc. C. E. (by letter).—This paper deals with more advanced topics, in connection with the control of National and State waters, than the average man is qualified to follow closely or to endorse in their entirety. Mr. Lewis does not write on these matters as a novice or a mere theorist, but as a responsible State official who has administered successfully the regulation of State waters for many years.

It is generally admitted, by those qualified to judge, that the water laws of Oregon are among the most carefully devised and effective of any in force in the United States. They contain certain admitted defects, more particularly in connection with the control of water-power, which may be expected to be eliminated during the next few years, as experience ripens; but the general provisions governing diversions, application to beneficial use, and appurtenancy to lands, are generally beyond serious criticism, except in details.

It is premature, of course, to claim too much for the Oregon system of water titles and control, their application being as yet quite young. Their ultimate value and success depend almost wholly on the capacity and integrity of the officials whom the State may appoint. Very few officials have had better experience than Mr. Lewis in water-right matters, or have devoted more energy and care to the numerous complex problems that are continually arising therefrom. He has been both an appointed and popularly elected State officer, having especial jurisdiction over the interests and prerogatives of the State in its control of State waters. All his official associations and connections necessarily have tended to emphasize in his mind the doctrine of State control of water. Therefore it is especially significant when a man of this type comes forward to affirm the importance and necessity of radical augmentation of Federal power in the control of public waters.

The Oregon Water Laws, like similar laws in other States, affirm the ownership by the State of all streams within the State, and establish rules governing their use; but an administrative State officer in charge of the practical application of these rules finds himself faced with apparently insoluble inconsistencies. Taking the State of Oregon as typical, there are the following complications: First, the National forest reservations, mainly occupying the great mountain ranges, in which most of the sources of the rivers lie, are under the exclusive and permanent administration and ownership of the United States. The appropriations and uses of water in these reserves, from every practical

standpoint, rest wholly with the United States, and not with the State, notwithstanding the doctrine of State control of all waters within the State. It will be readily understood by engineers that the Federal control of storage and diversions within these reservations necessarily conflicts with and renders partly non-effective any system of State control lower on the streams. Mr. Hopson.

Then again, there are the various reservations of reservoir sites and power sites under the Reclamation Act and for the conservation of water-power. These continually run counter to the obvious intent of the administration of the State laws controlling water use, because a State permit to use water for irrigation or power is useless in a large proportion of cases without the utilization of the sites embraced in the reservations.

Then, again, there are the indefinite rights of the Indian reservations, which in many cases are claimed to be superior to rights granted under the State laws to white settlers. These offer a complication not so serious as some, but certainly embarrassing.

Next there are interstate streams, which meander to and fro across the borders of the State, and on which adverse appropriations are made, first in one State and then in another, both being under absolutely independent State jurisdictions.

Then there are the navigable waters, under an indefinite control by the National Government. It is evident to any thinking person that control of the navigability of a stream necessarily implies some measure of control of the feeders of that stream. If the Government is to spend millions to deepen channels, build locks, dams, and other improvements, and be responsible for the maintenance of deep water for navigation, it is evident that it cannot permit private parties at their pleasure, even under State permits, to make diversions adverse to navigation interests. Hitherto, this question has not assumed the pressing importance that it will in years to come, because the population of the Western States and its utilization of water are still comparatively small; already, however, at certain points, there are unmistakable indications of a clash between the opposing interests of navigation and State-regulated appropriations. On one important California river, navigation is being seriously threatened by such diversions, and there is no body, either Federal or State, which has power to make an expert decision on the merits of the case in the public interest. In the meantime, important private interests are being permitted to grow up, with the sanction of the State and under the eyes of the Federal officials having charge of navigation, which inevitably must result in serious complications and losses in the future, but which both State and Federal officials are powerless to prevent.

The greater part of Oregon is within the water-shed of two great navigable rivers, the Columbia and the Willamette. On both these

Mr. Hopson. streams the Government is expending large sums of money in navigation improvements. The Willamette is an intra-state river; the Columbia carries drainage from all the Northwestern States, as well as from Canada. The summer navigation of both may be impaired to a marked degree when the various diversions believed to be feasible and desirable have been completed.

Serious differences of opinion are also arising from time to time over the uses to which inland interstate navigable bodies of water should be put, whether to be used for reservoirs, drained and reclaimed, or continued as mere navigable lakes. At present special Congressional action is necessary in each case, and is an uncertain and unsatisfactory recourse.

It is probable that in the majority of cases public interests will be best served by giving preference to diversions for beneficial use over use for navigation. The point of interest is that each case be considered on its merits at the proper time, and a definite policy be laid down by some competent authority, which obviously cannot be a creation of the State.

Mr. Cory. H. T. Cory, M. Am. Soc. C. E. (by letter).—This paper is much more than timely, and is exceedingly interesting, because, in the Western States particularly, the water laws are of vital importance, not only to very many engineers and individual owners, but to the general public. That there could prevail, in the United States, at the present time, such absurd conditions as exist in California and other Western States, is impossible to grasp fully until one comes into direct contact therewith. The notable exceptions are Wyoming and Oregon; and it seems probable that within a very few years most of the really Western States, if not all, will have improved matters greatly. Two years ago the California Legislature created a State Board of Water Control and a State Conservation Commission, and one year later changed the former to the State Water Commission. At the same time, and after the passage of an amendment to the State Constitution, it made a Public Utility Commission of the State Railroad Commission. As one result of the work and experiences of these various commissions, California will doubtless fall in line, within the next three months, with what is hoped will be the latest word in water-right legislation.

The final draft of the bill which will thus be presented to the Legislature by the administration in January, 1913, has not been made public. It is certain, however, that it will follow closely the Wyoming and Oregon precedents in general, that is, declare all water within the State to be the property of the public; distinguish clearly between the right to water and the right to use water; make beneficial use the basic principle—priority, purpose, period, place, and quantity; provide for determination and certification of existing vested interests,

and sane supervision, control, and certification of appropriations in the future. Due to the experience of the Railroad Commission, the provisions for reviewing the acts of that body, as well as numerous other features of the Public Utilities Bill, will no doubt be adopted almost *verbatim* for governing the work of the Water Commission. Mr. Cory.

The more important of these are as follows: A commission of five appointed by the Governor, and not elected or constituted *ex-officio;* five-year appointments, with provision for recall by two-thirds vote of the members of each branch of the Legislature; provision for investigation or hearing by any one commissioner and making every finding or decision of such single commissioner, when approved by the Commission, binding; power to determine existing rights and limit them to reasonable needs; power to order improvements for preventing water waste; power to subpœna and compel attendance of witnesses and take depositions; provision that all official documents and orders filed, when certified by a commissioner or the secretary under seal, shall be evidence in like manner as originals, and may be recorded; provision that no cause of action shall accrue in any Court unless application shall have been made to the Commission for a re-hearing, prior to the effective date of orders or decision; orders of Commission must be obeyed pending re-hearing, except as Commission may direct by order; power to modify orders after re-hearing; and, finally, that only the State Supreme Court may review any finding, time of appeal limited to thirty days after rendering of decision on re-hearing, but no new or additional evidence may be introduced in the Supreme Court, and the cause must be heard on the record of the Commission as certified by it. The review is limited to determination of whether the Commission has regularly pursued its authority, including a determination of whether the order or decision under review violates any constitutional right of the appellant—in other words, the findings and conclusions of the Commission on questions of fact are final and not subject to review, such questions of fact including ultimate facts.

Further, in the Railroad Commission Act, in the absence of a definite order by the Supreme Court, the Commission's order or decision is operative, pending final decision; and, if the Court stays or suspends the operation of all or any part of the Commission's order, such stay or suspension must contain a specific finding, based on evidence submitted to the Court and identified by reference thereto, that great or irreparable damage would otherwise result to the petitioner, and specify the nature of such damage, and require filing of a suspending bond before the Court's order becomes effective.

Lastly, it is provided that all actions and proceedings under that act shall be heard and determined in preference to all civil business, except election causes, irrespective of position on the calendar.

The Railroad Commission has, and the Water Commission doubt-

Mr. Cory.

less will have, the right and the duty to intervene in any civil action wherein the interests of the Commonwealth may, in its opinion, be concerned, and also initiate proceedings without being petitioned so to do.

It is provided that if any portion of the act be held to be unconstitutional for any reason, the validity of the remaining portions shall not be affected; and it is stated that:

"The Legislature hereby declares that it would have passed this act, and each section, subsection, sentence, clause, and phrase thereof, irrespective of the fact that any one or more sections, subsections, sentences, clauses, or phrases be declared unconstitutional."

The provision for making the Water Commission's decisions or orders binding, with appeal permitted only to the Supreme Court on exclusively legal features, is, in the writer's opinion, a very fundamental and far-reaching improvement over the procedure in Oregon or Wyoming. Several of the other features mentioned are distinct improvements, which though relatively minor, are nevertheless important.

The author's suggestion of equally stringent and positive National water laws and control, rather naturally, though quite unnecessarily, leads him to take up and discuss the water-power policy of the United States, which is practically synonymous with conservation, as politically understood. It seems to the writer that a National Water Commission, related to State Water Commissions in almost identically the same way as the Interstate Commerce Commission (in transportation matters) is related to the State Railroad Commissions, could be arranged, and fill the need exactly. General objections to "Bureaus in Washington" certainly do not apply to the Interstate Commerce Commission, and would have no weight with any other National commission clothed with powers like the California or Oregon Water Commissions or many, and especially the California, Public Utility Commissions. On the other hand, public sentiment, very effective control and regulation of public utilities, and conservation of the natural resources for the public by State authorities—especially in the "Progressive West"—have developed to such a point that it is beginning to be safe for the National authorities to begin to release their powers gradually to those of the individual States.

The essential thing is "Conservation", and whenever it can certainly be obtained by State authorities—and not an instant before—the National Government should relieve itself of the responsibility, because more flexibility and better adaptation of control to local conditions must result. The time is not yet ripe, and never may be, with water-power and forests, for example, but it is with some other things; and the civic awakening of the American public in very recent years is one of the most striking and significant phenomena of the present time.

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The author's suggestion as to the National Government developing power and supplying it at cost is exceedingly interesting, particularly because it was made at all. By virtue of their relationships, the members of this Society are probably—next to bankers and a few other types—the most politically conservative people in the Commonwealth. That a member should, in a paper before this Society, frankly and nonchalantly advocate, not only public but National ownership and operation of such a public utility as electric power, seems to the writer most significant of the rapid changes in progress throughout American society.

The writer feels strongly that such a thing would be unwise, and believes, with Mr. Dillman, that effective regulation, with a reasonable premium on able management, is preferable to public ownership, and would result in lower cost to the consumer; but, while that may be true in general, it is not always the case, and irrigation and the Reclamation Service is at least one notable exception. In opposing an idea nothing is truly gained by refusing to acknowledge antagonistic facts and arguments—usually, the effect is exactly the opposite.

The writer believes the conditions affecting the wisdom of Governmental construction and operation of power and irrigation projects in the West are fundamentally different. Power installations are almost invariably put in by well-managed corporations, and the markets are in large part now supplied by private aggregations of capital, and the effect on these by the Government supplying additional energy at cost, would be serious and really unjust. The U. S. Reclamation Service, on the other hand, always deals with projects which are entirely self-contained, and so do not affect the value of other privately or mutually controlled works, even if in the immediate vicinity. Furthermore, due to chaotic and most unwise laws, no well-managed private capital would deal with the irrigation situations comprehensively or wisely. Private irrigation projects have with very rare exceptions fared so disastrously throughout almost the entire West that it is almost impossible to secure capital for such work. Indeed, that is one of the chief reasons for improved State and National water-right laws discussed in this paper.

The Reclamation Service has since its beginning expended about $75 000 000 in irrigation construction, and is now working at the rate of about $1 000 000 monthly. During the past season (1912) it was ready to supply water for about 1 200 000 acres out of 3 000 000 acres for which the works are being constructed. The cost per acre ranges from $22 to, in one instance, $95, with an average of between $50 and $60. In few, if any, instances has any portion of such cost been paid as yet by the land owners, yet the selling price of land under the various projects—with the Government charges yet to be paid—has invariably increased, in some instances by more than $150 per acre. The works installed have uniformly been strictly first class, often monu-

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mental, sometimes possibly uneconomically so. In any event, the curse of Western irrigation—flimsy construction—has been avoided.

In short, the Reclamation Service has been doing a grand work, and if its operations were a measure of what might be expected of Government ownership as a whole, the proponents of such a policy would have a very strong, if not convincing, case. Its publicity department has been very efficient, but, on the other hand, it has gotten its result at a marvelously small outlay, and, even so, the people, for whom the primary purpose is supplying homes, are coming to the various projects all too slowly. This is almost entirely because the "back to the land" cry is a thing in which many join but few practice. The majority of the people, like the writer, prefer to live in the city.

The Reclamation Service has made not a few mistakes—it would be miraculous if it had not, considering the magnitude of its operations—and even many of its supporters feel that a frank admission of them would have made its position, among those whose opinions are worth while, stronger than it is. Indeed, the writer has urged that comprehensive and detailed papers, with complete cost data, be presented to this Society on each of the projects, preferably all at one time, to make a volume of *Transactions,* or a series of volumes, similar to those describing the new Pennsylvania Terminal in New York City. Opportunity to discuss matters would relieve the minds and hearts of those who feel that criticism is due, and, besides, the results of engineering experience of a Government department should be published in detail for the benefit of the Profession generally, and not kept within the confines of that bureau's staff. It is earnestly hoped that this suggestion will be followed.

Nevertheless, based on a fairly complete knowledge of only the Salt River, Yuma, and Orland Projects of the Service, and the observation and study of data examined during eight years with the Harriman Lines in California, Arizona, and Mexico—six years as a maintenance and operation official, with unusual opportunities to observe—the writer is convinced that, in these three projects at least, the Reclamation Service gets more actual work for a dollar than do the Harriman Lines. Such a statement, of course, is a judgment reached in a general way, because the character of the work done by the two organizations is so fundamentally different as not to permit of analytical detailed comparison item by item. Nevertheless, the writer feels very positive in this judgment and could cite very many instances, some minor and some fundamental, in support, though naturally many of the data at hand bearing on the matter could not properly be given out. Such a result is due to the fact that National politics has been eliminated from the Service, but still affects the Harriman Lines somewhat; that the Director of the Service has more authority and freedom in using his individual judgment than even Mr. Lovett of the railroads has, let

alone the local presidents; that the head officials, and consequently the general ideas of management of these railroads, have been changed, and the Director of the Service has not; that there is much less internal politics in the Service; and the Service organization and work, as yet at least, has not brought about the sharply drawn lines of cleavage between various departments and the leveling routine of "common standard" methods, channels of communication, limitations, prerogatives, etc., of the railroad. Mr. Cory.

The writer firmly believes that effective regulation of public utilities is greatly to be preferred to public ownership in general, and one important reason is the inefficiency of unwieldly organizations. The Reclamation Service is one of the smallest departments of the Government, and its efficiency is less than if it were smaller. A Government power service would be considerably larger, to develop any considerable percentage of unutilized water-powers, and immensely larger than any private corporation would ever be.

O. C. Merrill, Assoc. M. Am. Soc. C. E. (by letter).—There are certain fundamental questions in connection with water rights, the determination of which is necessary before a definite and consistent policy can be adopted or executed. Some of these questions have been brought to an apparently satisfactory solution in one or more of the Western States, and a more or less standard practice has arisen concerning them. Most of the so-called "arid" and "semi-arid" States, following the early lead of Wyoming, have provided means for the determination and adjudication of old water rights, for acquiring new rights, and for distributing the decreed water among the users. The most notable exception in this respect is California. One need not go far to find the explanation. Except for certain recent attempts, of somewhat doubtful value, to alter the statutory law of appropriation, California is still struggling along under a statute and a practice devised for and applicable to only pioneer conditions. Along with this has gone a line of decisions by the Courts constantly tending to saddle more firmly upon the State the outworn and inapplicable doctrine of riparian rights. Mr. Merrill.

It is true that the strict common law rule has been modified to the extent of recognizing the co-relative rights of riparian proprietors to the use of the waters of a stream, even to depletion. The Courts have also recognized rights acquired by direct appropriation, if at the time of the appropriation the lands below the point of diversion and riparian to the stream were in public ownership, a condition that at the present time is so nearly non-existent as to be negligible. The theory has been that at one time both lands and water rights were in public ownership; that the Government provided a procedure for the appropriation of the lands, and acquiesced in the appropriation

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of the water apart from the lands; and that, whenever riparian lands were appropriated under the public land laws, the full riparian right to the water immediately and automatically attached to the land, subject only to two limitations: first, the superior rights of prior appropriations of water; and, second, the co-relative rights of all other riparian proprietors. Priority of right to the use of water was made to depend on priority of time of appropriation, whether such appropriation was of water directly, or of land carrying with it a riparian right.

The only fully recognized water right by appropriation is one from a stream on which there are no private riparian lands at the time of the appropriation. When once a piece of riparian land has passed into private ownership, thereafter no right to divert water above such riparian land for use on non-riparian land can be acquired, except upon the acquiescence or neglect of the riparian owner—that is, by right of prescription. No right to divert water below riparian land can be secure against the possibility of complete depletion of the stream by upper riparian proprietors. This is the "California doctrine of appropriation." Moreover, even this is complicated by the existence of water rights in connection with the old Spanish land grants—rights derived neither by statutory appropriation nor under the common law, but in accordance with the civil law of the Spanish colonists.

The situation that exists is not one of mere academic interest; it has very practical consequences. Relying on the rulings of the Courts, certain landed interests in the San Joaquin Valley, for example, appear to have made a business of levying tribute on all appropriators of water from streams tributary to the San Joaquin River, regardless of whether the intended use had any effect at all on the amount of water which would reach the riparian lands. The writer has been informed of instances in which hundreds of thousands of dollars were paid to secure freedom from attack in the Courts, although the water was merely taken out of the stream, carried a short distance in cement-lined conduits and then returned to the stream. If the diversion had any effect upon the flow, it increased it by reducing evaporation and seepage losses between the points of intake and discharge. Even though the purpose of the appropriation be one that will not use up the water, as power development, a mere appropriator has so little standing before the Courts that it is deemed wiser to pay the tribute before litigation than to risk the payment of a higher tribute plus costs of suit after litigation. Riparian owners could hardly have been put into a better position had they been granted letters of marque against all appropriators.

Bad as the present situation is, the outlook for the future is still worse, unless the practice is changed. The prosperity of the State depends on its agricultural development. This will be accomplished in large part by breaking up the old Spanish grants and other large

land holdings—which have hitherto been devoted to stock grazing or grain growing—into small farms under intensive cultivation. The riparian right attaches to only such lands as border on a stream through a single continuous ownership. Riparian owners may prevent by injunction the diversion of waters to non-riparian lands. The breaking up of the large holdings means, therefore, the severance of large areas of land from any water right at all, and the concentration of water rights through riparian ownership into those lands, constantly decreasing in area, which border immediately on the streams—a menacing situation if left uncontrolled. Mr. Merrill.

The writer would like to be able to agree with Mr. Lewis that the case of Kansas *vs.* Colorado settled the question of riparian rights or established a precedent that the States will follow, but, in view of the decision of the Supreme Court of California on January 2d, 1909, in the case of Miller and Lux *vs.* Madera Canal and Irrigation Company (99 Pac., 502), he is constrained to believe that, as far as his State is concerned, the probability of relief from present conditions, or of being able to work out a rational solution of the water right problem, is becoming constantly more remote. The Court in this case said:

"The doctrine that a riparian owner is limited to a reasonable use of the water applies only as between different riparian proprietors. As against an appropriator who seeks to divert water to non-riparian land, the riparian owner is entitled to restrain any diversion which will deprive him of the customary flow of water which is or may be beneficial to his land. He is not limited by any measure of reasonableness."

It might very well be advisable to grant a certain amount of priority in time to the owners of riparian land, as such land, from its natural position, is likely to be more readily irrigable; but it is a violation of the common equities and of good public policy to allow riparian land owners to put to uneconomical use or to waste completely for an indefinite period one of a State's greatest resources, on the plea that sometime, somehow, they may find use for it. The riparian user of water should be restricted to precisely the same degree and measure of beneficial use as any other user. He should be given a certain time, under a statute of limitations, within which to make beneficial application of water to his land. After the expiration of that time, he should have only the standing of an appropriator in acquiring any additional rights to the use of water.

What is true of California is true to a greater or less degree of a considerable group of Western States, the so-called "semi-arid" States. In Washington, the Dakotas, Kansas, Nebraska, and probably in Montana and Oklahoma, the California doctrine of modified riparian rights has received the sanction of the Courts. Except in those apparently rare instances where appropriations were made when the

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banks of the entire stream were in public ownership, the only secure right other than a riparian right is one obtained by prescription—by open and notorious adverse user against the rights of the riparian claimant and without objection from him for the period prescribed by the statute of limitations. In any event, this is concluded to be the only form of secure title that a non-riparian owner can acquire in California, regardless of the fact that the State statutes are silent in respect to any other right than that acquired by appropriation.

Until this fundamental question concerning the acquisition of rights to the use of water is settled, either completely for or completely against riparian ownership, or in such a way as to make in all cases, riparian or otherwise, beneficial use within a reasonable time the basis of right to the use of water; unless the "dog-in-the-manger" attitude which the doctrine of riparian rights, modified or unmodified, upholds, is definitely and completely repudiated, no hope of building up an enduring policy with respect to the public control of water resources, or of securing a complete utilization of them, or of giving definite title to them, can prove anything but illusory. What possibility is there, for example, of making any final adjudication of water rights on a stream where some users claim by appropriation, others by prescription, and others by virtue of the riparian situation of their lands? The quantity of riparian lands is constantly changing. Each riparian owner has as good a right as every other. An adjudication of such rights made to-day is inapplicable to-morrow if any of the conditions as to the volume of stream flow or amount of riparian land has changed. No effective policy could be established concerning the acquisition of new rights, for such rights would be automatically acquired in certain cases by the acquisition of title to the land, in other cases they would be obtained by prescription due solely to the negligence or lack of interest of lower riparian owners, and in other cases by purchase or condemnation. Real appropriation, the securing from the State of a clear title to the use of a definite quantity of water, cannot exist in such a situation. Similarly, but little force can attach to any scheme for an administrative control of the distribution of water. No user, save he who has acquired a prescriptive right, has any claim to the use of a definite quantity of water, or a quantity sufficient for a definite area. Everything depends on the co-relative rights of the other riparian owners under the conditions of the moment. In a country dependent on irrigation, the doctrine of riparian rights leads to nothing but unending confusion. If the Courts of Oregon have allowed that State to join the roll of those which have repudiated such a doctrine, then Oregon is to be congratulated.

Even if this first problem, the right of use, should be solved satisfactorily, there will still remain certain other fundamental issues, some of which have been met in several States, but others of which, as

far as the writer can learn, have not been met in any State. Without attempting to name the States which have taken action or the means which have been used in the solution of certain of these problems, the writer will merely outline what seems to him to be the fundamental questions involved in the administration of water rights in those States which have not deprived themselves of the power to name the conditions on which water rights may be acquired and retained. Mr. Merrill

Before a thorough-going public policy can be developed for the administration of public waters, it is first necessary to determine what is the underlying purpose to be accomplished, and then to provide the means for its accomplishment. The underlying purpose should be to encourage the settlement of lands, the development of agriculture, and the building of homes and communities—the promotion of the general welfare of the citizens as a whole, rather than the special profit of certain individuals. To accomplish this purpose, it is necessary to consider carefully what character of right should be granted under varying circumstances, where or in whom the ultimate ownership of such right should vest, and what limitations, if any, should be made, and what conditions imposed upon the grant.

The purpose expressed above, however often lost sight of, has actuated the disposition of the agricultural lands of the public domain. These lands have been given to those who would cultivate them and who desired to make homes on them. It has been deemed contrary to a wise public policy to allow the title to large bodies of public land to pass to one owner. The limit has been placed at that amount which an individual owner could reasonably be expected to improve and cultivate. To have given more would have resulted in withholding the excess land from use or, if the patentee sold it, in increasing the cost to the purchaser and user over what the latter might have acquired it for first hand had the excess area been left open to entry. In securing patent to public land, the individual must act for himself. He cannot obtain title as a member of an association or of a corporation. Although details differ, the same general practice of encouraging individual enterprises and of preventing speculation and non-use has also been applied to the disposition of public mineral and timber lands. Recently, however, it has come to be realized that, although this principle applies to the disposition of agricultural lands and certain kinds of mineral lands, there are other kinds of mineral lands to develop which with reasonable economy is far beyond the reach of ordinary individual enterprise, and that a new public policy and a new method of disposition must be provided for them.

In granting title to the use of public waters to the extent of beneficial use, the practice of water appropriations has followed, in general outline, the practice in respect to the appropriations of agricultural land; but in making little or no distinction between the

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appropriator and the actual user, the general practice has been quite different. To accomplish the purpose which was expressed above, the title to water rights should vest in the ultimate user. The character of right which the appropriator should be allowed to acquire should depend on whether the right applied for is to be used by the applicant himself for his own purpose, on his own property, and only indirectly for profit, or whether it is to be used by the applicant directly for profit in the delivery and sale of water or in supplying the public with some commodity or service which will be derived from or be dependent on the use of the water appropriated.

When water is appropriated by individuals, by mutual associations, or by municipal corporations for irrigation purposes on their own lands, where any profit which may be derived from the undertaking will be indirect through the products of the soil, or where these same classes of users have appropriated water for their own domestic purposes, the character of the right granted to the actual users should be that of ownership limited by two conditions: (1) that the water shall be applied to a beneficial use without unnecessary waste; and (2) that the water shall be appurtenant to the land or the locality for which it is appropriated.

The principle underlying the disposition of public agricultural lands will apply also to the use of water for manufacturing purposes or for power, if such use is confined to localized industries and to power developments comparable in extent with the agricultural operations of the homesteader; but, to give title of ownership to the use of water for manufacturing or for power in operations of the magnitude which customarily obtain, would be at variance with the general principle which has been recognized in the disposition of the public lands. Such uses should rather be treated in accordance with the special policy proposed for disposition of extensive areas of mineral lands. The title which the appropriator may acquire for power or manufacturing purposes, in any event for such of them as comprehend quasi-public uses, should be something less than ownership. Some such lesser title should also be granted the appropriator for irrigation or for domestic use when the appropriator is not the user but is engaged in a business which takes on a quasi-public character.

The primary object in the administration of water rights for irrigation, namely, to encourage the settlement of lands, the development of agriculture, and the building of homes and communities, will best be promoted if the water rights are in all instances made appurtenant to the land on which the water is used. Whether the appropriation is made by a mutual association, by an irrigation district, or by a person or corporation engaged in the business of delivering water to the land, the water right should be an appurtenance of the land irrigated, and the actual owners of the water right should be the land owners

themselves. Appropriators of water for sale should not be given a water right equivalent to ownership, but only a right of agency—a license to act in behalf of those who are the real owners until such time as they may desire to assume complete control. Mr. Merrill.

Similar considerations hold with respect to the appropriation of water for domestic uses. Such water rights also should be appurtenant to the lands or the localities served, such as the homestead of the individual appropriator, the district embraced by a mutual association or other organization, or to the territory included within the boundaries of a municipal corporation; and the actual owners of the right should be the users themselves. It is manifest that, in general, the right cannot be made appurtenant to specific parcels of land or other property, and that the ownership cannot be individual, as with irrigation water rights. Either the individuals must act through organizations, such as municipal corporations, or must receive the service through the agency of persons or corporations who will make the appropriations and perform the service. If the second alternative is adopted, the appropriators should be allowed to acquire from the State only the necessary right of agency—a mere license to act in behalf of the users until such times as the users may desire to act for themselves—while the real ownership should remain in perpetuity in the community served.

From the point of view of a public administrative policy, one of the most important uses of water is for the development of power. In the form of electric energy, this use extends over wider areas, reaches a larger population, and finds a more diversified application than any other use of water. Moreover, on account of the magnitude of the undertaking and the character of the services performed, organizations which handle it are almost invariably corporate, and the business itself, with few and minor exceptions, assumes a quasi-public and monopolistic character. Clearly, this use of water cannot, in general, be made appurtenant to any specific parcels of land or to any definite locality, because the use may go beyond even the boundaries of the State. Nevertheless, the appropriator, because of the quasi-public character of his business, acts only as the agent of the public which he serves. This agency should be recognized in the grant, and the appropriator should be given only a license or lease, leaving the real ownership in that widest public which is represented by the State itself.

With a complete title, or even one limited to beneficial use, the appropriator will invariably capitalize the right to its full commercial value, if no more, regardless of whether he paid for the right or received it gratis. It is not likely that any Court will refuse to allow full value for a water right of such character in any proceeding respecting the valuation of the properties of the appropriator. If so

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valued, then the appropriator would be allowed to earn an income thereon, and that portion of the public served would be called on to pay it in the form of rates. Furthermore, if it should become desirable for the public to purchase the properties of such appropriator, it would be necessary to pay, in addition to the value of all other properties, the full market value of the water right which the public once owned but gave away.

Every person or corporation engaged in a business of this character should be allowed to earn a fair, even liberal, return on the capital actually and legitimately invested, but no such person or corporation should be allowed to acquire property gratis from the public and afterward charge rates and earn an income on it. Such a situation can be avoided in part by retaining title to water rights and by giving the appropriator only a license or lease. It can be avoided wholly only by reserving in addition the right to cancel such license at any time by paying full value for all the physical property of the licensee. If the purchase is made before the licensee has had a reasonable time within which to realize the expected profit from his undertaking, a bonus in the form of a certain percentage of the value of the physical property, varying inversely as the time which the license has run, should, in the writer's opinion, be also paid. It is believed that the investment of the licensee would be fully protected if, in the place of intangible values, a bonus of three-fourths of 1% of the appraised valuation of the real property should be paid for each year less than fifty years that the license has been in force. Such a license would be in effect an indeterminate franchise—the form which best protects the interests both of the public and of the investor.

If water-right applicants are given licenses for fixed terms without the reserved right of purchase, all such licenses or grants will, as any other unexpired franchise, have a value in proportion to the time that the license has to run, unless the license is granted on the express condition that no value shall at any time be assigned to or claimed for it, or the rights granted under it, in respect to any valuation of the properties of the licensee for the purpose either of rate making or of sale.

The appropriators of water for distribution and sale, whether for irrigation or for domestic use, and the appropriators of water for power purposes are on a common footing. Each will be engaged for profit in a business which is quasi-public; each will use in his business partly his own property and partly the property of the public; each, in his use of the public property, should be considered as the agent of the public. This relation between the quasi-public appropriator and the public can be fully maintained only by:

(1) Making water rights for irrigation purposes appurtenant to the lands to be irrigated;

(2) Making water rights for domestic purposes appurtenant to the lands or communities served; Mr. Merrill.
(3) Retaining in general public ownership water rights for power purposes;
(4) Granting licenses terminable by purchase to quasi-public appropriators of water, whether the use be for power purposes or for purposes of sale and delivery of water to others for domestic supply or for irrigation;
(5) Granting such licenses only upon condition that they shall not be capitalized for any purpose.

To accomplish the objects and to carry out the principles named above has been the chief purpose of the Federal Government in its administration of water-power sites on the public lands. The use of the public land for water-power development, both within and without the National forests, is open to all applicants who can give any assurance of a purpose actually to develop the sites. Mere speculation in power sites has been discouraged, and properly so. Only one who has been personally in touch with the situation can realize the extent of pure speculation in water-power rights. Such speculation is no assistance toward actual devolpment; it is rather a hindrance. No sooner is it learned that a municipality or an operating company is planning to make developments, than water-right locators and right-of-way applicants immediately appear, trying to forestall the municipality or corporation and hold them up for a price. The Oro Electric Corporation, of California, has recently been working up plans for additional generating stations to be located on streams within the Plumas National Forest. Within a few days after the facts became known, applications for the use of the site were filed in the District Office of the Forest Service at San Francisco for the sole purpose of securing a prior right in order to demand from the corporation a price for withdrawing. The attempts were not successful. This, however, illustrates what has come to be an established industry in some of the Western States. It is much more successful with relation to water rights than to rights-of-way across the public lands. It is the duty of any administration, State or National, to check such practices. Whatever price the speculator may secure is so much more to be added to the capital of the operating company, and the public pays interest on it forever. Just why this indirect method of getting money out of the public pocket finds so many ardent defenders it is hard to say. The ultimate effect is no different than if it were taken from the public treasury. One method adds to direct, the other to indirect, taxes; but in either case the public pays the bill.

William G. Davies, Assoc. M. Am. Soc. C. E. (by letter).—It is particularly pleasing to note the general optimism in regard to public rights, and especially Mr. Johnston's view that: Mr. Davies.

Mr. Davies.

"Because the States have been slow in asserting the doctrine of public rights in property which is essentially of a public character, we need not fear that the public has lost title in any way."

We are impatient at the lagging of water legislation so far behind physical development and need. At times it does us good to curb our impatience and note conditions in other countries.

While France was a Roman province, it had the Roman water laws. Under the Merovingian kings, property was held under a different tenure, and all rights of ownership in watercourses and waters were vested in the rulers themselves. In feudal times the watercourses came largely under the feudal lords, but control was constantly disputed by the king.

Upon the decline of feudalism and the beginning of modern history, about 1500, Roman law was largely restored. Modern history in France began then with navigable and raftable streams as public domain, and smaller streams entirely under the control and virtual ownership of riparian owners.

From this position, of almost complete ownership and control by riparian owners, the centuries have seen them gradually lose control. They no longer have much, if any, ownership of the stream-bed, until dry; they have largely lost their rights to the water, no longer having the right of ownership, this being now the possession of the Nation, but having only a right to use the water, which right cannot be forfeited by non-use.

In the United States—California for example:

True riparian rights doctrine—the right of a riparian owner to have the waters of a stream pass his property "undefiled in quality and undiminished in quantity"—has fallen. We now have the modified riparian rights; and we well know, from the history of France, Italy, and Spain, and from the strong present tendency to minimize individual rights and magnify public rights, that this modification will be a continued process, and we will end with riparian rights being nothing at all, or, at most, but a shadow of their former selves. Even now, in California, leading attorneys declare the best water right to be one acquired by prescription—adverse use.

Mr. A. E. Chandler points out* that, based on the Court decision of Anaheim Union Water Company *vs.* Fuller (150 Cal., 331), riparian rights attach to only the least parcel of land abutting on a stream, and that, if an owner of a tract of riparian land subdivides and sells part of his land, those portions not contiguous to the stream lose their riparian rights unless the conveyance declares the contrary, and that the land thus severed can never again regain the riparian rights by any reconveyance.

* "Elements of Western Water Law," published December, 1912.

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Now we have the right of "reasonable use"; but when we have a water right based on modified riparian rights and "reasonable use", there is but little difference in net results, so far as the irrigator is concerned, between this and appropriation and reasonable use. The principal difference is that, under the Oregon administrative system, the water right is readily determinable, ownership being practically as easy to determine as land ownership, whereas, under the riparian rights system, the holder is never free from the worry and expense of a lawsuit about his title to water. Some authorities have pointed out that passing a law annulling riparian rights existing before the passage of the act is unconstitutional. Nevertheless, the passage of an act annulling riparian rights not asserted at the time of passage of the act would be of use, as it would influence Courts in the constant minimizing of riparian rights.

This process took centuries in France, and it is not yet complete. Are we doomed to spend an equal time? No; for, besides the fact that the world moves faster now, we have rapid hydro-electric development as a stimulant for rapid legal development.

When a water right is for an individual irrigator or for water-power for a single grist mill, it is of much less general public importance than when it is for development of power for a public utility electric power company with a network of lines covering some great district. The economic interest of the public in a river is vastly increased by hydro-electric plants of large public utility companies. This being so, it will inevitably and necessarily follow that public rights on that river will increase and individual rights adverse to the general public good will decrease; and this is as it should be. On every hand we see the disposition to enlarge or define more clearly and defend public and community rights and minimize individual rights.

Mr. Justice Cooley, in defining a public use, as the term is applied in the law of eminent domain, said:

"The reason of the case and the settled practice of every government must be our guide in determining what is or is not to be regarded as a public use, and it can only be considered such where the government is supplying its own needs or is furnishing facilities for its citizens in regard to those matters of public necessity, convenience or welfare which, on account of their peculiar character and the difficulty of making provision for them otherwise, are proper, useful and needful for the government to regulate."

From this definition a public use depends on times and conditions. A private property to-day may become a public necessity to-morrow. A stream of interest to but a small group of irrigators to-day may speedily become of very great interest to a whole valley using electric power.

In California the electric power development has been rapid, the merging of power companies rapid, and, in consequence, a conservation

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commission was appointed to report to the 1913 Legislature. It has done a very great deal of work in preparing reports. Because of the arousing and focusing of attention, the chance of getting some good water laws enacted is much better than for many years past, when irrigation interests alone were at stake.

This same general condition, however, prevails throughout the Nation. The Eastern States were not at all interested in irrigation, and, before hydro-electric power development began, they could get along very well with their old laws based on riparian rights doctrine. The arid States felt the pinch of need, and so kept constantly changing their water laws. The two portions of the Nation—the humid and the arid—were getting more and more dissimilar in their water laws; but, now that hydro-electric development is growing so rapidly, there once more appears a reason and a basis for fairly similar water laws. This is no doubt the thing that must be seized on when we urge that there should be essential uniformity of water laws among the States and abrogation of riparian rights.

There is an instance of such rapid economic change and consequent rapid legal growth in Switzerland. In that country each of the twenty-five cantons had always exercised the fullest authority over unnavigable streams within its borders. This worked very well until the development of water power on a large scale began. Then the necessity for change of laws became apparent to many. In 1891 the Swiss Society "Frie Land" issued a petition demanding the insertion of the following clause in the Federal Constitution:

"All the water powers of Switzerland not yet utilized are the property of the Confederation. A federal law shall regulate all that concerns the application of this resource and the distribution of the net benefit produced by it."

In 1895, after a thorough investigation, the Federal Assembly rejected the proposition and approved the following resolutions, which sound amusingly familiar in America:

"1. The great majority of Cantons which possess and exercise the right of sovereignty over their watercourses and which often derive direct income therefrom are not disposed to renounce that right of sovereignty.

"2. In general, the public interest in the matter is safeguarded by the Cantons according to the importance of each case.

"3. The transfer to the Confederation of the right of sovereignty and of administration over watercourses would not afford any advantage to the country and would not present any source of new revenue, either to the Confederation or to the Cantons.

"4. The possibility of availing itself of the water powers in the future could very easily be reserved to the nation by careful legislation.

"5. In general, the Cantonal authorities are better able than the Federal Government to see to the development and utilization of water

powers, both from an economic point of view and from that of public and industrial interests. Mr. Davies.

"6. The importance, sometimes limited, of the object in question and the securing of a rational use of the various falls, which is often difficult, require the co-operation of the Cantonal authorities specially qualified to protect the development of the local trades and industries of the Cantons."

In the years following 1895, hydro-electric development went on apace and public opinion kept an equal stride, so that in October, 1908, by a popular vote of almost 6 to 1, the people completely reversed the former decision and adopted the following amendment:

"The Federal Congress shall have supervision over the development of water power.

"The Federal Congress shall make provision for the disposition of water-right concessions, shall prescribe the terms thereof, and shall regulate the transmission and distribution of electrical energy so far as may be necessary to protect public interests and to provide for the proper development of such resources.

"All water rights to which the terms of the federal law do not extend shall be under the jurisdiction of the Cantons, which shall dispose of the concessions, regulate the same, and impose taxes and fees for their use, but such regulation taxes and fees shall not be so severe as to prevent or inhibit the development of water powers.

"The National Government shall regulate and dispose of concessions for powers located on inter-cantonal and national boundary streams, and shall determine the taxes and fees to be imposed thereon, after hearings have been granted to the Cantons interested, but such taxes and fees shall be collected by the Cantons.

"No power development in a stream located within the Union shall be transmitted to a foreign country without the consent of the Federal Council.

"The Provisions of the Federal law shall apply to water rights concessions already existing, except in cases specifically exempted therefrom by law."

Now, to come down to the present and to America: Public opinion in America is by no means ready for any such Federal law as that just quoted from Switzerland. The growth and change of public opinion here will not take the centuries it took in France for the modification of riparian rights, nor yet will it be so rapid as to take only thirteen years, as in Switzerland, for the great legal change noted. A few days ago Secretary of the Interior Fisher, in his Annual Report to President Taft, urged a "definite and comprehensive water-power policy for streams on the public domain and navigable streams not on the public domain." While good, it is not comprehensive enough, no doubt intentionally so, as it does not provide for water rights for other things than water power, nor specifically for water rights on interstate streams.

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The question is: What is the limit of Federal or State legislation now practicable or now probably possible?

Fourteen years ago,* it was stated that:

"An adequate remedy [for difficulties on interstate streams] through national legislation, it is to be feared, is out of the question * * * Any general legislation by the National Congress relative to water rights would mean an upheaval too great and too far-reaching to be practical. It would mean that a number of the States would have to make changes, not only in their laws, but in their constitutions, and would overturn the long line of established practice and decisions of the courts in dealing with this question.

"The waters of natural streams within the boundaries of the several States in the irrigated regions being, by virtue of constitutional provision, by the abandonment of control by the General Government, or by usage or the decisions of courts, the sole property and under exclusive control of these States, it is clear that reform of evils which exist by reason of defective legislation in these States, or by reason of lack of all legislation, can only be effected by proper legislation on the part of the legislative branch of the governments of these States. It is clear, too, that reform can not be effected unless the several States, in forming new legislation or correcting that which already exists, take into consideration the rights of the citizens of neighboring States as well as the rights of their own people."

A convention to consider uniform laws was urged. Several conventions under various auspices have been held. The U. S. Reclamation Service has worked out an outline form of law for adoption in the arid States. There has been a great growth in uniformity, and but little, if any, growth in diversity of water laws.

In the fourteen years since the foregoing opinion was written, there have been three great events: First, rapid development of hydroelectric plants, intensifying the need of better water laws and nationalizing the problem. Second, a great increase in the interest taken by the Federal Government in the use of water in both irrigation and power development, and a relative growth in the legal powers and claims of the Federal Government as compared with that of State Governments. So great has been this change of thought that in the opinion of Mr. Morris Bien, of the U. S. Reclamation Service, "It is difficult to find any possible grounds for the theory of State control, as to non-navigable streams within the State, which are not tributary to a navigable stream." Third, a great growth in public opinion to the effect that a much greater number of our difficulties of all kinds should be settled by administrative processes than by appeal to Courts.

The writer proposes the following as a compromise measure, but one which accomplishes the object of limiting water rights on inter-

* *Bulletin No. 70*, Bureau of Irrigation Investigation, U. S. Department of Agriculture, entitled, "Water-Rights Problems of Bear River" (an interstate stream), published under the direction of Elwood Mead, M. Am. Soc. C. E.

state streams in accordance with beneficial use, and making the water right appurtenant to land rather than personal property. Mr. Davies.

Pass a Federal law with provision as follows:

Provide for a permanent bureau for interstate streams and lakes; that this bureau shall have no jurisdiction except on interstate waters (and possibly on navigable rivers and on non-navigable streams on the public domain); that this bureau shall make surveys for each interstate stream, determining areas of land irrigated, area irrigable, size of ditches, points of diversion, volume of water used, discharge of stream, storage sites, power sites, dates of appropriation and use, etc.; that this bureau shall prepare a table describing land irrigated, giving date of priority, maximum amount of water allowed for irrigation of each tract, nature of use, allowable dates of use per annum, as does the State Engineer of Oregon on an intra-state stream, but ignoring State lines; that the division of water is to be based on beneficial use and the greatest good to the greatest number; that no lawsuit involving interstate right to use water on a stream may be begun in a Court until the bureau has made its survey and table of priorities, and then only on questions of law, not of fact.

There is undoubtedly much greater hope of passing such a law as this than one under which the Federal Government would take control of all streams.

Mr. Chandler states* that decisions on interstate waters to date have been mostly on small streams. In Bean *vs.* Morris, the U. S. Supreme Court, May 29th, 1911 (221 U. S., 485), said, concerning the waters of Sage Creek, an interstate stream:

"We believe that it always was assumed, in absence of legislation to the contrary, that the states were willing to ignore boundaries, and allowed the same rights to be acquired from outside the states that could be acquired from within. * * * The doctrine of appropriation has prevailed in these regions [Montana and Wyoming] probably from the first moment that they knew of any law and has continued since they became territory of the United States. * * * Before the State lines were drawn, of course, the principle prevailed between the lands that were destined to be thus artificially divided. * * * The only reasonable presumption is that the states, upon their incorporation, continued the system that had prevailed theretofore, and made no changes other than those necessarily implied or expressed."

In a case between riparian owners in California and appropriators in Nevada on West Carson River, the Court did not attempt to ascertain individual priorities, but attempted an equitable adjustment between those in one State as a whole and those in the other State as a whole.

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* "Elements of Western Water Law."

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These decisions being so fair, the question arises, "Why substitute an administrative body for a Court in dealing with interstate streams?" And the answer must be: "For the sake of efficiency."

Mr. Freeman.

W. B. FREEMAN, ASSOC. M. AM. SOC. C. E. (by letter).—This valuable paper and the discussions thereon have been particularly interesting to the writer, because, for a number of years, he has been working in Wyoming, Colorado, and New Mexico, and has been intimately connected with matters pertaining to water rights and the distribution of the flow of the rivers of those States. At present there is no question of more vital public importance in the United States than the equitable distribution of the waters of interstate streams, involving a definition of the relation of the States to each other and to the Federal Government.

The Oregon law, as outlined by Mr. Lewis, is an excellent one. The fact that it is working well is rather strong evidence of its merit; and it is gratifying to know that there is a successful code of water laws in actual use, for there have been many defects in all previous ones, including that of Wyoming (which, heretofore, has been regarded as the model), and, in many States, the laws—or lack of laws—in existence have been utterly ineffective and confusing. The writer believes it to be desirable that all States—at least the irrigation States—should proceed immediately to adopt new laws, fairly uniform in character, with the Oregon law as a standard; then, as Mr. Lewis clearly demonstrates, the matter will be practically taken out of the Courts, where only litigation and confusion has been the rule, and put in the hands of a Board of Control capable of determining, classifying, and indexing every right in existence. In that way it will be possible to know the quantity of water available in each stream system, for future appropriations, and the conditions under which such water is available, without detriment to prior beneficial uses. That depends, also, of course, on the variation of stream flow from one season to the next; but, with discharge data available, an expert board is surely able to make a better division of waters than the Courts.

Oregon Laws.—In the discussion, several suggestions have been made for the improvement of the Oregon law. One, which is very good, is that the unit of measurement be defined more clearly. For instance, instead of stating that the maximum use is 1 sec-ft. for 80 acres, it would be much more definite to state that not more than $2\frac{1}{2}$ or 3 acre-ft. per season can be used for each acre irrigated, and it would also be well to define the maximum flow allowed in the ditches under each right. In that way much trouble would be avoided in the division of water between storage and direct users, and there would be no question as to the extent of each right, regardless of the length of the irrigation season.

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Another writer suggests that the rights and duties of the State Engineer be defined more specifically in certain matters, such as his authority to grant additional rights on a stream until its flow and the characteristics thereof are well known from a record extending over a number of years. It is quite necessary that general rules be laid down for his guidance, but it would seem to be impracticable to make laws relating to minute details in matters of this kind (nearly as much so as to attempt the regulation of precipitation and run-off by statute), and a great deal must necessarily be left to the discretion of the State Engineer and the Board of Control. They, however, should be given special authority for non-approval of filings and applications pertaining to wildcat projects, or schemes of which the merit is doubtful.

On the whole, it seems that the Oregon law needs very little improvement, and, with slight modifications, there is no reason that it should not be as applicable in the humid as in the arid or irrigation States. In fact, many of the Eastern States have already discovered the benefits to be derived from irrigation, and, if one is to believe the reports of the Department of Agriculture, the practice may be extended to every part of the Union. The present uses of water in the Eastern States, where it is returned to the stream largely diminished in quantity, do not really differ materially from use for irrigation, and, of course, power use in both localities is identical. The difficulties in regard to the adoption of uniform laws seem to exist largely in the minds of those who have been educated to riparian doctrines and can conceive of use on no other basis.

Interstate Streams.—It is very evident that there is need for immediate and comprehensive legislation regarding the division of interstate waters. The present system, whereby the Courts, at intervals of a number of years, render decisions in specific cases, some of which are contradictory in character and not one of which has yet clearly outlined the relation of the States, is utterly without efficiency, and the result is that matters are constantly becoming more complicated as the appropriations continue to increase. Congress, which had the power to create an Interstate Commerce Commission, has certainly some power to legislate without fear of being overruled by the Courts in a matter which the Courts themselves have apparently been unable to decide in a satisfactory manner.

The case of Kansas *vs.* Colorado is the one most usually referred to in discussions on this subject, and the decision itself and the parts thereof have been given a thousand and one interpretations, many of them diametrically opposite, by the adherents of various sides of the question. The people in Colorado, for instance, are generally of the opinion—by virtue of the fact that the State Constitution makes the State the owner of all waters within its boundaries—that it is the privilege of the citizens of the State to divert every drop of water, if they

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wish, from streams flowing into neighboring States, regardless of prior appropriations in those States; and they constantly refer to the result of this suit and a decision which they believe virtually sustains their contention. It is rather hard for one who is impartial, and has been trained in the doctrine of priority and beneficial use, to conceive that any Court of Justice should support such a theory, and there is serious doubt whether such was the intent of the Supreme Court. However, it is largely on that idea that the Colorado interests expect to win the case of Wyoming *vs.* Colorado, now before the U. S. Supreme Court.

Mr. Lewis demonstrates that the Kansas-Colorado decision practically asserted that the "doctrine of riparian rights, * * * is not the law, and, therefore, never has been the law" even of interstate waters. Mr. Knowles, in his discussion, shows just as clearly that the "principle of equal rights, which is the basis of riparian rights," was followed. The writer, however, believes that the learned judges, although following the language of riparian rights—because, from education and precedent, they knew no other—actually had in mind the doctrine of prior appropriation and beneficial use. In fact, it is difficult to imagine, from the conditions surrounding the case and the evidence introduced, how riparian issues could be considered; therefore, the arguments advanced by Mr. Lewis are perhaps well grounded. The whole matter, however, shows very plainly how unsatisfactory and confusing is the present method of administration by injunction and Court decisions, and that effective remedies are necessary.

The Government and the States.—Another issue to be decided is the relation of the various Federal bureaus to the States, in the administration and use of streams, intra-state and interstate. Several who have discussed this paper have taken advantage of the occasion to attack the Reclamation Service and its work, and Federal policies in general, relating to irrigation and the conservation of natural resources. This is hardly pertinent to the question; neither is it the writer's intention to defend the bureaus thus attacked. The Reclamation Service, regardless of its unpopularity in some quarters, has done a great work and has given a tremendous impetus to irrigation development, and the Conservation movement, which seems to be a good one, has been handicapped by insufficient Congressional legislation and because the powers and duties of the officials in charge have not been defined.

The present methods of administration of the Reclamation Service and other bureaus, and the grievances resulting, are traceable directly to the chaotic state of affairs resulting from a system based on government by injunction and the determination and adjudication of water rights by the Courts. In the absence of definite limitations of their powers, it was necessary for these bureaus to define their own, and it is safe to say that many which they have assumed are not constitutional

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It is pointed out by one writer that the Federal Government recognizes the sovereignty of the States in matters pertaining to the use of water for irrigation purposes because the Reclamation Service files its applications for water rights in the offices of the State Engineers and in accordance with the regulations of the State where the project is located. This is hardly consistent with the fact that this Service has made filings in a number of instances for all the unappropriated waters of a stream, a right which an ordinary appropriator might find it difficult to sustain. Nor does it seem entirely fair, as has been done in the case of the Rio Grande, a stream rising in Southern Colorado and flowing into New Mexico and Texas, that the Reclamation Service, not only assumed the right, some ten years ago, to file on all the unappropriated waters of the stream and its tributaries north of Elephant Butte for the irrigation of land in Southern New Mexico, Texas, and old Mexico, but has been able to maintain it by rulings of the Secretary of the Interior, and thereby prevent further development in Southern Colorado and Northern New Mexico; whereas it is questionable whether the Reclamation project will be completed, for any considerable percentage of the ultimate acreage, within fifteen years of the time that the filings were first made. That proposition and the policy involved certainly seem to be at least as unfair and as much open to criticism as the one advanced by Colorado of the right of its citizens to use every drop of water within the State. It seems easily possible to improve on both of these rather arbitrary viewpoints or methods of procedure, to the end that justice be done to every one.

Administrative Control.—The foregoing illustration is given without prejudice, either to the rights of a State or a Federal bureau, but as an example of the results of the present system. Such a condition could hardly have existed if there had been a State and National system of administrative control, as suggested by Mr. Lewis, with the authority of the Commission of Control clearly defined; and, in the light of all the experience obtained, there seems to be no reason that such a Commission should not be able to operate expeditiously and successfully. It certainly should be more efficient in securing results than the workings of the present system would ever lead one to hope for.

The personnel would probably consist largely of technical experts in matters of this kind, and provision would no doubt be made that States involved in a particular controversy be given ample representation by their own authorities to act in conjunction with this Commission in the settlement thereof. When the number of interstate streams in the United States is considered, or even those in the irrigation States, and the number of appropriations on each stream along its course, it would seem that there will be sufficient work to occupy the attention of such a Commission for a long time.

Mr. Freeman. The writer sees no more equitable principle to follow, in the adjudication of a stream from source to mouth, than the doctrine of priority and beneficial use, regardless of State lines, there being a clear definition of domestic and other preferred uses, and the fact also being remembered that there will be a certain return flow to the stream from the upper users. All this will undoubtedly require a great deal of study and expert investigation, but it seems inevitable that such adjudications must be made eventually, and the sooner it is done the more advantageous it will be. Experience does not lead to the belief that much relief can be expected if the Courts continue to adjust these matters.

The first move would seem to be the adoption by the States of uniform laws, similar in character to those in Oregon. As soon afterward as possible there should be a complete determination and adjudication of all intra-state water rights, with their priorities, by the State Engineer and the Board of Control of each State. Such a movement in the States would no doubt accelerate the creation of a National Commission, which, when formed, would have the benefit of all data as to water rights of the various States, and this would expedite the complete adjudication of the interstate streams and the settlement of controversies relating thereto. The writer believes that engineers will agree that results of that kind are necessary and desirable, whether or not they agree on the method of procedure suggested. It seems certain that the Engineering Profession must have much to do with the final determination of these matters, and the more they are decided on their merits as technical rather than legal questions, the more satisfactorily and expeditiously can they be handled.

Mr. Lewis. John H. Lewis, Assoc. M. Am. Soc. C. E. (by letter).—The foregoing discussion has greatly strengthened the writer's conviction that it is the duty of the Engineering Profession to lead in the reform of State and National water laws. There is no profession which is better qualified to judge as to what the laws relating to water ought to be, and no organization better fitted than the American Society of Civil Engineers to carry on a persistent educational campaign to secure the ultimate enactment of such laws. Much of Oregon's success in the administration of her water laws is due to the pioneer work along this line in Wyoming. However, on one or two points, in his admirable discussion of the principles of law, it is believed that Mr. Johnston, former State Engineer of Wyoming, is in error.

He says that "The States have not properly administered the smaller streams which they own." From this it is assumed that Mr. Johnston means to say that:

"The waters of all natural streams, springs, lakes, or other collections of still water within the boundaries of the State of Wyoming are * * * the property of the State,"

as provided in Article VIII, Section 1, of the Constitution of Wyoming. This, on its face, looks like an irrevocable grant by Congress of all waters within its borders to the State of Wyoming, and that the United States, or any adjoining State, can have no jurisdiction or control over the same. Mr. Lewis.

Later, Mr. Johnston states that principles are of so much more importance than details of law that he does not feel inclined to discuss the question of State or National control.

It seems that this question is fundamental. It is true that the principles of priority and beneficial use, as a basis for water titles, should be understood and adopted by both the States and the Nation, but, until we can decide as to whether this system of water titles should be administered by the States or by the Nation, there is little use discussing details.

It is doubtful if the approval by Congress of a lengthy document containing the words above quoted will ever be construed as a grant of water to a State, as to do so would virtually give to Wyoming the control of all Government land within its borders, as this land is worthless without water. Such construction would be in conflict with Section 3 of the ordinance forming part of the Wyoming Constitution, providing that the State disclaims all right and title to the unappropriated public lands lying within its boundaries.

Few, if any other, States have such positive declarations as to State ownership of water. Whatever the law in the Eastern States, there must be some unappropriated waters in most streams subject to public control under any law that may be enacted at this time. It is important that the public ascertain and determine the location and extent of these public waters so that flood-water reservoirs may be constructed for the preservation of life and property during periods of excess run-off, and for the benefit of navigation, power, irrigation, and domestic supplies during the low-water period. To do this we must have some sort of an interstate or Federal administrative system, corresponding in character to that found desirable within the State for such work. The riparian owner cannot insist that those destructive floods continue to flow uninterrupted as in the past. The right to the use of all water in every part of the United States which has been put to beneficial use can in general be considered as vested. In some parts of the East, the riparian owner has perhaps some rights in the ordinary or low-water flow of the stream, notwithstanding the fact that he has not put the waters to beneficial use. The public is entitled to know the extent of all vested rights, of whatever nature, and to have provided by law some orderly method of acquiring rights in such unappropriated water, so that development can keep pace with modern demands. Knowing the extent of such rights, they can be pur-

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chased or condemned, and their value to the owner will be increased accordingly.

If we had not discussed the relative merits of county control and State control in Oregon, we would still be attempting to administer streams through a cumbersome system of county control. It is high time that all who are thinking along conservation lines get down to a discussion of the fundamental questions on which any constructive legislation must be built, that is, the proper jurisdiction for administrative purposes.

Mr. Knowles states that "There may be a question, also, as to whether the law has become sufficiently determined to justify the creation of an administrative commission," also, "If it be true that the law for all cases has not been determined, it would appear almost certain that any important contest between States, before an administrative board, would be appealed to the Supreme Court, until the universal law had become established, and a Commission might therefore be of no value at present."

We have practically no interstate or National law relating to the administration or distribution of streams. How, then, can the law ever become settled by judicial interpretation? In the absence of such statutory laws, the Courts, as new cases arise, must practically legislate on the point at issue in order to decide the matter. If Mr. Knowles expects to await the building up of a Federal administrative system by this slow and expensive system of Court legislation, then we may as well abandon the whole subject of constructive legislation. Many lawyers take this view of the question, as the chaotic condition of State and National laws is desired by a certain class of powerful corporations seeking the monopolistic control of water without use, and because litigation of this character extends over many years, and is therefore highly profitable to the legal profession.

Appeals from any administrative board established by Congress to regulate streams are certain to be carried to the highest Courts. The quicker such appeals are taken, the better. If the law is defective, it can be amended, and we will be on the high road to a solution of these many important problems. If it is upheld, such point will probably never be carried again to the Courts. A few such cases will settle all doubtful points, and the administrative board will proceed with its work, speedily and economically, and with great benefit, through the higher development and use of our natural resources.

With reference to the Oregon system of water titles, Judge Bean, of the United States District Court for Oregon, said, in a recent case (Silvies River case, 199 Fed., 495):

"I am impressed with the soundness of the view that a proceeding for the adjudication and determination of the rights to the use of the waters within the state, instituted and conducted as provided in the

legislative act of 1909, is in effect a proceeding on behalf of the state through an administrative or executive board to have judicially settled in an economical and practical way, the rights of various claimants to the use of the waters of a stream or source of supply, and thus avoid the uncertainty as to water titles and the long and vexatious controversies concerning the same which have heretofore greatly retarded the material development of the state." Mr. Lewis.

The statement by Mr. Dillman concerning two strong corporations on a stream in California which "shut out the small fry" by protracted litigation over water and then withdrew the suit and divided the water between themselves, is a good example of the kind of justice some States mete out to their citizens. By arguing the question of States' rights or Federal rights thus delaying action, we will soon find "the small fry" getting the same kind of justice meted out to them in the Federal Courts through litigation over interstate rights, irrespective of the fact that in some States good water codes may be in effect.

In the above-mentioned case, construing the Oregon administrative code, one of California's most powerful corporations was shut out of the Federal Court and was compelled to submit to the administrative officers of the State. The same tactics had been pursued for years on Silvies River, in Oregon, where litigation has been in progress for so many years that most of the "small fry" were about ready to sell out for little or nothing and leave the country. They now have some chance to have their cases tried without ruinous expense, and once for all time, which is the important point.

Mr. Dillman wishes to be delivered from any more Government Bureaus, but if only a Government Bureau can protect "the small fry," then by all means let us have another bureau. California, however, needs a State Water Bureau worse than any other State in the Union, and the lack of such bureau is probably due to the presence of such powerful companies having control of great quantities of water under lax laws which do not require beneficial use as the basis of the right.

The writer is glad to note the admission by Mr. Sheley that the attempt to adjudicate water rights in Utah through proceedings in the Courts has been found to be impractical. This is the history in every State where an earnest attempt thus to adjudicate water rights has been made, and is the reason why many States are now turning over all water matters to administrative boards having only subordinate judicial powers.

Mr. Knowles believes that, by applying the "rule of reason" to the two conflicting doctrines of riparian rights and appropriations and use, both can be merged into one and the conflicting laws of adjoining States be thus brought into harmony. This plan has been attempted for years in California. It has resulted in the most confused jumble

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of impractical laws to be found in any State in the Union. What the law is in that State no one can tell without examining thousands of pages of conflicting Court decisions, and then no two investigators of the law can agree exactly as to what the law really is. The rule of reason applied by one judge will differ from that applied by another. In those large interstate or National problems like the control of the Mississippi or Ohio Rivers, where millions of dollars of the peoples' money must be invested, we must have some absolutely definite legislation before such work is undertaken. Such works should endure for all time, and the necessary title to water should not be left to the reason or logic of any judge, however learned.

In his discussion, Mr. Knowles has attempted to point out certain errors in law made in the paper. As Mr. Johnston says, "Principles are of so much more importance than details of law," and the writer believes it is almost a waste of time for the engineer to discuss such detailed matters. Therefore, no attempt will be made to point out what are believed to be errors in his reasoning.

We must recognize the fact that we have at the present time no interstate or National administrative laws relating to the diversion and use of streams. It should be the province of the engineer to ascertain what the law ought to be. When this has been accomplished, and such law is framed in harmony with the laws of Nature and in conformity with good engineering and business practice, then legal experts should be called in to advise what changes are necessary to conform with good legal practice. Some of the best lawyers are always found on the side of reform to promote the public good. Owing to the present confusion of water laws, it will doubtless be found that almost any law for the highest public good can be sustained in the Courts. We are certain, however, that every section of such law which conflicts in any way with vested property rights will be pointed out by the lawyers in Congress, and most fully corrected before any such legislation will be enacted. However, it is believed that Congress will welcome suggestions from the Engineering Profession as to what the law ought to be, in order to stimulate the development of vast engineering enterprises.

Divided Control.

Since writing the paper, the Department of the Interior, of the United States, has issued regulations concerning rights of way through the public lands and reservations of the United States * * * for Power Purposes, approved August 24th, 1912. These regulations apply to Interior Department lands, and are much the same as those adopted by the Agricultural Department for lands within the National Forests. They are based on the theory of land control, impose certain limitations on the use of water, and amount to the same thing as the United States controlling the water. Permits are revocable at will by

the Secretary, and terminate in fifty years unless renewed under certain conditions. Mr. Lewis.

These regulations have confirmed the writer's opinion that the United States controls not only the public lands in the Western States, but also the water flowing over the same, and that the State exercises almost no control whatever over such waters, except by virtue of these regulations which require compliance with State laws. When such lands pass into private ownership and water is appropriated, then such water to that extent is subject to State regulation.

The Acts of Congress of 1866 and 1877 are believed by many to be an irrevocable grant of water to the States, but this, apparently, is not the case.

Concerning this question of State control of waters on Federal lands, the Supreme Court of the United States held, in the case of the United States *vs.* Rio Grande Dam and Irrigation Company (174 U. S., 690-703), that:

"Although this power of changing the common law rules as to streams within its domain undoubtedly belongs to each state, yet two limitations must be recognized: (1) That in the absence of specific authority from Congress, a state cannot by its legislation destroy the right of the United States, as the owner of lands bordering on a stream, to the continued flow of its waters; so far at least as may be necessary for the beneficial use of the Government property; (2) That it is limited by the superior power of the general Government to secure the uninterrupted navigability of all navigable streams within the limits of the United States."

"In other words, the Court holds that the jurisdiction of the United States over the natural watercourses upon the public domain" is superior and paramount to the jurisdiction of any State; and that all needed measures may be taken by the Government to preserve the watercourses of the country for "at least" the two purposes named above "even against the action of any state," in authorizing, under its laws, appropriations to be made. The Court, especially in the Kansas-Colorado and Rio Grande cases, clearly intimates, to say the least, that the Government might also make other claims to the water than for its use for irrigation, or as a riparian owner. Whether it will do so, time alone will tell.*

"The Government is still the owner of the surplus of the waters flowing upon the public domain, or rather the owner of all the waters flowing thereon, remaining after deducting the rights to the use of the same which have vested and accrued in some legal way to individuals and companies. * * * It therefore follows as the result of the ownership by the United States of the waters flowing on the public domain, that any dedication by a State of all the waters flowing within its boundaries to the State or to the public, amounts to but little, in the

* Kinney, in his recent work on Irrigation Law, p. 1096.

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face of any claim which might be made by the Government, at least to all the surplus, or unused waters within the State."*

Kinney states, further, that Congress, if it had seen fit, could have laid claim to the necessary surplus water for projects constructed under the Reclamation Act, by "virtue of the fact that title to the surplus waters flowing over the public domain is in the Government," and not under the State laws, as was provided in Section 8 of such Act. Much confusion within the States may result eventually where Congress has reserved water for Indian reservations, which, after many years, has never been used by the Indians, but, in the meantime, has been used by others below.

It appears, therefore, from these authorities and from the wording of the Acts in question, that Congress, if it saw fit, could repeal at any time existing laws relating to water and adopt some new law for all surplus and unappropriated waters on the public domain. In such an emergency, we would have divided control within the Western States to such an extent as to make any control impracticable except through the closest co-operation.

Mr. Cory states that it is beginning to be safe for the National authorities to start gradually releasing their water powers to the individual States, and that National ownership and operation of such a public utility as electric power would be unwise.

The writer will attempt to show that in some cases the first proposition is unwise, and that the second proposition is the only logical plan for the development, distribution, and use of water power in the Western States.

Transfer to States.

Owing to the conservation sentiment in the Eastern States, it is believed to be very unlikely that the United States will ever turn over to the States, for administrative purposes, the control of public lands now withdrawn for power purposes. We cannot await the settlement of this complicated political issue before undertaking some of the large enterprises in the West, which per unit cost are far cheaper to construct than many of the small projects now being undertaken by private capital, or by the public under the Reclamation Act, or Carey Act, without co-operation with all other interests in the field. To illustrate the point, one large irrigation project and one large power project in Oregon will be described, and an attempt made to show that immediate construction is what is wanted, leaving to the future the settlement of the question of State or Federal control of streams.

Deschutes Irrigation Project.

On the Deschutes River, at Benham Falls, 181 miles measured along the stream from its junction with the Columbia River at Oregon, a

* Kinney, on Irrigation, Second Edition, p. 692.

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75-ft. dam will back the water up for more than 20 miles, flooding 25 000 acres, and will store 700 000 acre-ft. of water. The remainder of the 1 053 000 acre-ft. of water which passes this point annually can be diverted from this stream in the vicinity of the Town of Bend, without storage, for irrigation during the summer, and will supply about 100 000 acres of land. The stored water will irrigate about 210 000 acres in addition. The entire stream, at Benham Falls, will thus supply 310 000 acres, which is equivalent to a strip of land 1 mile wide and 484 miles in length. It has been roughly estimated that the cost of construction will be about $30 per acre.

Incidental to this development, an enormous amount of summer power will be produced, for, in the 37 miles between the dam site and the last diversion dam for irrigation purposes, there is a total fall of 1 300 ft. During the height of the irrigation season, about 5 000 sec-ft. of water will be available for power development in the upper portion of this stretch. As the different diversion dams for irrigation are passed, smaller quantities of water will be available for power until the last diversion at Cline Falls is reached, where about 1 500 sec-ft. will be diverted for the lower districts to be irrigated.

During the winter, when the reservoir is filling, the power possibilities along 70 miles of the stream channel, having a total fall of 2 600 ft., will be destroyed, but this is not a serious loss, as the seepage water below the reservoir can be carried through the lower irrigation canal to a vertical drop about 800 ft. in total height, where more winter power can probably be developed than can be used for many years to come in that section. Near the lower end of this district which it is proposed to irrigate, the stream enters a deep rock-walled cañon from which it is impossible to divert water. Some of the largest tributaries enter just below the section to be irrigated, so that, with full storage at Benham Falls, about 4 500 sec-ft. can be depended on at the mouth of the stream, provided 80 000 acre-ft. can be stored on Crooked River, one of the upper tributaries, to piece out the low-water flow. There is a fall of 1 400 ft. in 111 miles of this lower river, below the mouth of the Metolius River, its principal tributary. On this latter stream there is a fall of about 2 600 ft. in 70 miles. Practically all of this enormous fall can be utilized. The flow of the stream is most peculiarly uniform, due to the light porous pumice stone formation which covers the entire upper drainage of the Deschutes and its principal tributaries to a considerable depth.

These enormous power and irrigation possibilities are so interrelated that the United States could not surrender the power sites without jeopardizing its land and irrigation rights. The United States owns about one-half the irrigable land in this district, and has withdrawn enough land along the stream practically to control the power situation. So enormous are the power possibilities that ultimately

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much of the Government land along the Columbia River and other streams, both in Oregon and Washington, within economical pumping range, will ultimately be reclaimed by power from the Deschutes River.

To settle finally all the complicated State and National questions, so as to permit the logical development of this stream for the greatest good to the greatest number, would require many years. In the mean time, the railroad which is now constructed up the Deschutes River to the Town of Bend, will have been constructed southward through the proposed reservoir site where its right of way is now partly cleared. Also, numerous power plants will have been constructed along the main stream immediately below the proposed dam site, thus preventing the storing of the winter flow. These and other like complications, if not prevented, will accumulate to such an extent that in a few years this magnificent project will be too much encumbered to undertake; but all such difficulties can be prevented without in any way retarding present development, if such private development is forced to conform to some public plan for the stream as a whole.

The development of much of the water power on the Lower Deschutes River has been greatly complicated by the recent building of a railroad along either bank of the stream so near the water surface that the construction of dams to an economical height is made impossible without moving the grades. These roads were forced to elevate their tracks around the proposed power sites of two private companies, also one site selected by the U. S. Reclamation Service. With but little, if any, extra cost, these roads could have been constructed at a uniform height of 100 ft., or more, above the bed of the stream. The water power, however, belonged to the public, and we have been so busily engaged in disputing the right of the State or the Nation to control that, in this instance, much power has been almost completely destroyed through public neglect. The same will occur with respect to the irrigation and power possibilities of the upper valley, where only 65 000 acres are now under irrigation, and where there is a possibility of irrigating 500 000 acres if the water is put to use in accordance with a comprehensive preconceived plan.

The State has a large measure of control over the waters of this stream, and the United States, through its National Forest Reservations, Indian Agencies, and water-power withdrawals, exercises a very large control over such stream through the control of land rights. Therefore, the public interest could be well protected if the State and the Nation could unite in some workable plan for the greatest good.

Co-operation.

On February 21st, 1913, the Legislature of the State of Oregon appropriated $50 000 for co-operation with the U. S. Reclamation Service in the preparation of a comprehensive plan for the Deschutes

River Basin, and the Secretary of the Interior has agreed to allot a like sum for such work, which is to be carried out jointly. Mr. Lewis.

The State law authorizes the withdrawal of all necessary water rights for the protection of the project. On the completion of the plans, any part or all of the project can be turned over to private capital for construction, on payment of the full cost of preparing such plans, the money to go into a revolving fund for the promotion of other projects; provided that no such project, or any part thereof, shall be turned over to private capital except on full hearing of all interested parties, and proper assurance that public interest will be safeguarded.

Anticipating difficulty in securing private capital to carry out so large an undertaking, the Legislature has recently submitted a constitutional amendment for the vote of the people, authorizing the issuance of State bonds to the extent of 2% of the assessed valuation of the State for the construction of irrigation and power projects, and for clearing and developing the cut-over timber land of the State.

In case it is found necessary for the public to undertake the development of its own resources, it is being strongly urged that the State co-operate with the Nation, rather than attempt a new organization. It is believed that the people will not vote millions of dollars for development projects without tying up to some stable, experienced organization which has actually made a success in this important work. The few mistakes of such service only serve to strengthen the arguments for co-operation, as these mistakes are not likely to be repeated. In this way the difficult question of politics and patronage can be overcome. It is believed that the United States can be prevailed on to co-operate with the State, dollar for dollar, as $20 000 000 has already been voted by Congress for irrigation in the West, without any offer of co-operation on the part of the States most vitally interested.

It thus appears that the policy most likely to secure immediate development is that of co-operation between the State and Nation, leaving to the future the discussion of States' Rights and Federal Rights. If we should take the position, advocated by Mr. Cory, of demanding the turning over by the United States to the States, of its various resources in the West, we might never attain that end. If we did, we would then be many years behind the times, as each State would have to organize a complicated technical bureau to handle the work, and would inevitably repeat many of the mistakes of the United States, if not do far worse.

In a very brief campaign, we have actually attained in Oregon co-operation with the United States. Furthermore, the people are in entire sympathy with a comprehensive development programme, as the Legislature has just appropriated $450 000 to complete one unit of the big Deschutes scheme, where the State has made a failure in

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the reclamation of 27 000 acres from Tumalo Creek, under the provisions of the Carey Act. Water rights to about 20 000 acres of land had been sold where the supply was barely sufficient for about 3 000 acres.

Power Development by the Public.

Each house of the Oregon Legislature has passed a somewhat different bill appropriating $25 000 for the thorough investigation of the Columbia River Power Project, near The Dalles, as outlined by the State Engineer in his Fourth Biennial Report. It is believed that an agreement will be reached within the next few days and the investigation authorized. It is the purpose of this bill to make diamond-drill borings, prepare detailed plans, specifications, and estimates of cost for a 536 000-h.p. project, and to estimate the cost of transmission of this power to various points in Oregon and Washington; also to estimate the cost of distributing such power from sub-stations in the cities to individual consumers. To show the saving which would result, estimates are to be furnished giving the cost of producing power from other sources than water. This work is to be carried out jointly by the States of Oregon and Washington, and the United States, if co-operation can be had from such sources. A joint legislative commission from both States, headed by the respective Governors, has examined the project and recommended the plan. In order to show why private capital cannot undertake the construction of this project, and why it is necessary for the public to promote it, and perhaps eventually construct it, a brief description of the entire plan is presented.

Columbia Power Project—Summary.

Installed Machine Capacity: 536 000 h.p. with 300 000 h.p. of 12-month power and 236 000 h.p. of 8-month power.

Location: At Big Eddy, 3 miles above The Dalles, 90 miles east of Portland, the metropolis of Oregon, and 200 miles from the mouth of the Columbia, and the Pacific Ocean.

Market: 240 000 h.p. for fertilizer works, the remainder for the iron and steel industry, wood distillation plants, aluminum, carborundum, carbide, alkali works, electro-chemical industries, woolen mills, pulp and paper mills, light, heat, and power in wholesale blocks to encourage new industries, and without competition in retail business of local power companies.

Dam: Present channel of Columbia to be closed by a dam, 300 ft. long, approximately 180 ft. high above foundations, and new channel excavated in solid rock 1 400 ft. wide, and water surface regulated by removable dam, sections 100 ft. long, 60 ft. high. Location, 1½ miles above power-house.

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Power-House: Oregon or Washington side. Planned for Washington side, at Big Eddy; 1 200 ft. long, about 200 ft. high above foundations, contains 21 turbines, supplied by canal, 300 ft. wide, with 20 ft. depth of excavation, 1½ miles long from pool above dam.

Head of Water: One-half height of Niagara Falls (73 ft. at low water; approximately 42 ft. at high water). Natural fall at low water in river 11 ft., with fluctuation of 90 ft. at proposed dam site.

Water Available: Minimum, 50 000 sec-ft., maximum, 1 390 000 sec-ft., mean flow, 235 000 sec-ft. Drainage area at The Dalles, 236 800 sq. miles. Stream flow record for 33 years available.

Turbines: Runner 16 ft. in diameter, shaft of steel 30 in. in diameter, 60 ft. long, supporting generator on top approximately 36 ft. in diameter, maximum capacity, 32 000 h.p., all at 80 rev. per min. on one oil bearing. Maximum water capacity of each, 5 000 sec-ft.

Cost: $23 076 000, or $77 per h.p. for 300 000 h.p. Cost of low-tension power station, $6.90 per h.p. per annum. Cost of power by steam from waste sawdust, about 0.5 to 0.7 cents per kw-hr., or from $33 to $46 per horse-power-year. Minimum price at Niagara for large blocks, $9.00, average about $15.00.

Transportation: Competing transcontinental railways at power-house, with navigable water from same to Pacific Ocean, and for many miles inland.

Raw Products: Abundant and accessible for many different industries. Air, 20 000 000 tons of nitrogen above 1 sq. mile of the earth's surface, or enough to supply fertilizer for the world for 50 years. Timber, iron ore, lime, salt, and other products accessible because of rail and water transportation.

Market for Manufactured Products: The World; as the power plant is at the gateway to an inland empire, with water outlet to the Pacific. With public docks at Portland, and the Panama Canal completed, shipping facilities will be available for industries seeking world markets.

Low water in the Columbia River invariably occurs during the winter, and floods during the summer. These floods are caused by the melting of snow in the high mountains at the head of the stream.

Fig. 6 shows the total power available in the Columbia during an average year. It is based on approximately maximum head conditions, and made up from records of the last 10 years, with an assumed turbine efficiency of 80%, a generator efficiency of 93%, and on the

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assumption that sufficient turbines are installed to use all the water at all stages.

The curve, $B_1\ B_2\ B_3$, in this diagram shows the power which could be delivered to the switch-board by twenty hydraulic units, assuming the generator large enough to transform the power without overheating.

The line, *DD*, is the normal rated generator capacity, 400 000 kw., or 536 000 h.p. The lower of the three lines at any point is the maximum power output of the station. Thus it will be seen that at low-water periods the flow of the river limits the station capacity ($A_1\ O$ and $R\ A_3$); at intermediate discharge, the generator capacity limits ($O\ P$ and $Q\ R$); and at high-water periods the turbine capacity is the controlling feature ($P\ B_2\ Q$). The resulting minimum capacity is represented by the shaded line. The line, *CC*, is the adopted minimum station capacity of 300 000 e.h.p., or represents the line of perennial power, sometimes called "primary power," or that which is deliverable with reasonable certainty under all conditions.

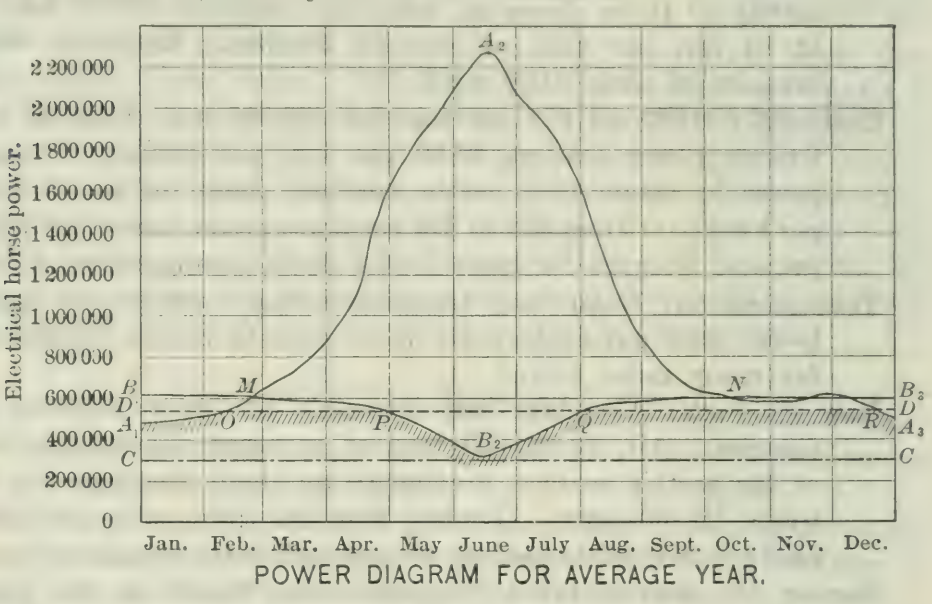

FIG. 6.

The shaded line, at nearly all places, is far above this line, *CC*, indicating that much surplus power would be available at certain seasons of the year if customers could be found whose operations could be temporarily suspended during floods and during extreme low water.

Thus it will be seen that, during practically all of February, March, April, August, September, October, November, and December, or 8 months of the average year, a total of 536 000 e.h.p., or 236 000 e.h.p. of surplus or secondary power, could have been delivered to the switch-board without any additional capital investment.

During the summer irrigation season of nearly 5 months, as much as 500 000 h.p. in addition could be developed at an initial cost of

about $44 per h.p. and furnished at about $4.50 per horse-power-season. This power could be transmitted along the river for pumping to arid lands in both Oregon and Washington. It is this power that the United States should be interested in; and this is the reason that the Reclamation Service should co-operate in the investigations. (The writer has been assisted in working out the technical details of this project by Messrs. L. F. Harza and V. H. Heineking.) Mr. Lewis.

The proposed dam will flood about 6½ miles of the navigation canal which is now being constructed by the United States at a cost of several million dollars. The lower 1½ miles, containing three locks, will be utilized in connection with the proposed power project, the boats entering the stream immediately above the dam and proceeding up the wide open river. At low stages, 10 ft. of water will be found above Celilo Falls. Owing to this saving in construction cost, the ease with which boats can go up the wide river, in place of the long, narrow, shallow canal, it is believed the United States should not only co-operate in the investigation of the project, but also in its construction.

When this navigation canal was projected there was no prospective market for 536 000 h.p. To-day it is believed that the entire quantity could be disposed of within a short time after the completion of the plant, and to new industries not now located in the Northwest. The construction of this project would mean the investment of perhaps $100 000 000 of new capital in this district, besides affording profitable employment to thousands of laborers, and add greatly to the taxable wealth of the State.

This statement relative to market is based on correspondence with large power users in different parts of the world. The representative of certain foreign capital has examined all power possibilities on the Pacific Coast, including this one. After going over the proposed project, as briefly outlined above, he agreed to recommend to his company the purchase of 240 000 h.p. at $9 per horse-power-year, delivered at the low-tension bus-bars of the generating station, putting up a $200 000 surety bond, provided the State would contract for 40 years, the power to be used in making fertilizer from the air.

This sale alone, amounting to $2 160 000 per annum, would fully finance the entire project for the development of 536 000 h.p., paying interest, depreciation, operating and maintenance expenses, carefully estimated at $2 068 000. It is probably true that some of the remaining power could be disposed of at from $30 to $50 per horse-power-year to the railway lines in this vicinity, but only a very limited market could be found at such figures. This one plant could supply a city thirteen times the size of Portland to-day, or one of nearly 3 000 000 inhabitants. Forty years hence the population of Portland may reach this figure.

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NECESSITY FOR CO-OPERATION.

One-half the bed of the stream, and of the water, is subject to the control of either State, and the United States controls all matters relating to navigation.

Private capital would have extreme difficulty in negotiating with these three agencies, and would run considerable risk through subsequent legislation detrimental to their interests.

The Oregon Legislature recently came within a few votes of imposing a heavy annual license tax for regulation purposes on all vested water-power companies, and the Washington Legislature has now under consideration a bill providing for the forfeiture of water rights in that State if electricity from a plant now under construction is transmitted across the river for sale in Oregon.

Oregon now limits franchises to the use of water for power purposes to 40 years, and imposes an annual tax of from 25 cents to $2 per horse-power-year, depending on the percentage of power appropriated which is put to beneficial use. Partly for this reason, power plants prefer to locate just over the line in Washington. The United States would probably insist on some different franchise limitation, and the payment of heavy charges to the Federal treasury for revenue or for the improvement of the stream. The State charges alone, or the annual payments to the Federal treasury, would defeat this project, as there is even yet no market for this power except at extremely low prices.

The limit for fertilizer works is, perhaps, $9 or $10; for iron and steel works, $7; for aluminum, alkali, carborundum, and carbide works, $10; and cost or less for heating purposes. Much power could be used in heating retorts for the distillation of waste wood products, in each cord of which, as stated on reliable authority, there is $18.40 in value, as follows:

4 gal. of turpentine, at, say, 30 cents	$1.20
85 lb. of calcium acetate, at 12 cents	1.70
2 gal. of alcohol, at 50 cents	1.00
40 gal. of tar	8.50
1 000 lb. of charcoal	6.00
	$18.40

The utilization of this waste wood would thus greatly stimulate the development of our already extensive lumber business, and the making of pig iron in the electric furnace would afford a market for the charcoal thus produced.

Pig iron is now being produced successfully at Heroult, Shasta County, Cal., where a plant has been in operation for the past six years. It is stated on good authority that the experimental stage of

this industry is definitely past, and the recent enlargement of the plant appears to confirm this fact. Mr. Lewis.

Electrical metallurgists, such as Heroult, are now very confident that, when the new method is installed on a large scale, it will be feasible to attain a yield of 12 kg. of pig iron per horse-power-day. Under such conditions, current at $9 per horse-power-year could compete with $4 coke.

At such rates there could be but little if any profit for private capital in the electrical end of the business, and few enterprises which have but recently passed the experimental stage can afford to risk $23 000 000 in a power plant in order to get a small block of cheap power. The present retail power rates in this vicinity vary from $74 to $460 per horse-power-year, and thousands of horse-power in the State can be developed and delivered to consumers in reasonable quantities at perhaps $15 to $25 per horse-power-year. With the opening of the Panama Canal, it is believed that the development and sale of power by the public at little, if any, profit, will do far more to stimulate industry and promote the peace and general prosperity and welfare of the people in the Western States than can ever be accomplished through irrigation, and that, without such public development and distribution of power, this development will be forever denied to this section of the United States.

For the same reason, therefore, that the United States was compelled to go into the irrigation business in order to dispose of its arid lands, and for the same reason that the State is now planning to cooperate with the United States in the irrigation and colonization of both public and private lands, so the State and Nation must go into the power business if its untold wealth of water-power resources is ever to be put to use. In Oregon, more than 3 000 000 h.p. is going to waste in the streams, to produce which in steam engines would consume $144 000 000 worth of coal annually. This economic waste is enormous. Why should Oregon impose heavy taxes on power development and the companies pass this on to the people, thus maintaining power prices the same as in coal-producing regions, when, by basing the price on cost of construction, power could be sold at such low rates as to give the western section a tremendous advantage over the eastern section which has enjoyed for years an unfair economic advantage over the Northwest in having cheap coal?

The capital now invested in the exploitation of the natural fertilizer deposits in Chili is $136 000 000. It would require a capital investment of $860 000 000 to assure the production of an equal quantity of artificial fertilizer from the air by means of the electric furnace, assuming that cheap power were available.* It is proposed that Oregon

* Special Agents Series No. 52, p. 76, U. S. Dept. of Commerce and Labor.

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adjust her laws so as to compete with the world in attracting these new industries.

It makes little difference whether the State or the Nation backs the bonds for irrigation works, as long as such works are built speedily and economically, and the lands are colonized so that the money is returned with interest. Likewise, it makes little difference who owns the land or the water at feasible power sites, so long as these sites are developed economically and the power put to use. Co-operation for the speedy construction of these projects should be the aim of all, leaving the control to the State or the Nation in proportion to the funds advanced. It is a waste of time to talk about State or National control at this stage. These matters will all be adjusted later.

TERRITORY AFFECTED BY COLUMBIA AND MISSISSIPPI RIVER POWER PROJECTS.

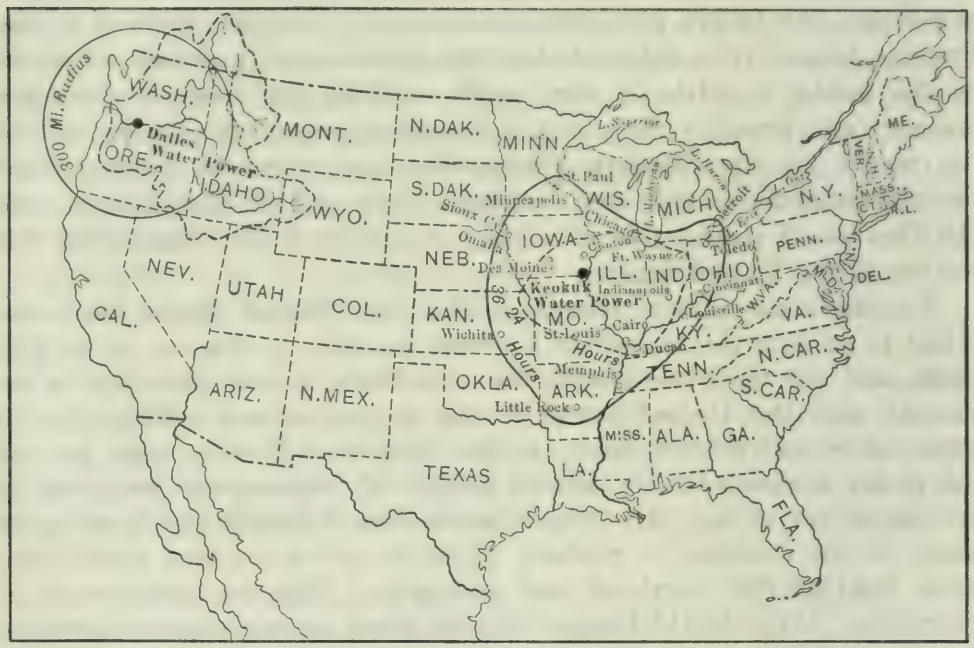

FIG. 7.

ADVANTAGES IN CO-OPERATION.

There are many advantages in co-operation. From Fig. 6 it will be noticed that approximately 2 000 000 h.p. can be developed in the Columbia near The Dalles, during June of each year. When the Columbia River is at low stage, almost all the other rivers in the Northwest are at high stage. Power from this plant could be transmitted, from 300 to 400 miles, at prevailing commercial rates, if no other cheap power were available. From Fig. 7 it will be observed that the entire Northwest falls within a radius of 300 miles from The Dalles. The demand for power in this territory varies from hour to hour, and from season to season, with as great fluctuation as stream

flow. By having all plants in this section tied into one system, supplying all consumers, a tremendous economic advantage will result. No individual State can ever accomplish such a comprehensive scheme, nor could the United States acting alone. Any independent action, by either State or Nation, will result in endless complications, such as are already beginning to be felt in the Northwest through the divergent laws and interests of different States. Mr. Lewis.

Private capital is aware of the economies attained by consolidation, but the people favoring public owership and control are many years behind the times. Each town is attempting to supply its own citizens for special uses. As a result, from two to four times the necessary machine capacity must be installed to carry peak loads, and the plant is practically idle for a large part of the time each day. The cities, perhaps, can take care of local distribution, but the States and the Nation should take care of the generation and transmission of power.

The information on Fig. 7, relative to the Mississippi plant at Keokuk, was furnished by the Keokuk Industrial Association, and is intended to show the very great territory affected by cheap power. According to this authority the 200 000 h.p. to be developed at the Keokuk plant will cost $125 per h.p., in comparison with $77 estimated for the Columbia River plant. The difference in cost prices at which power could be sold from these plants will, for some products, more than equal the freight rate from Portland to Chicago.

In the Eastern States, where water power is somewhat limited, and where all the people cannot enjoy equal advantages from the distribution of power at low prices, it may be found to be better policy to impose annual charges on water-power companies, so as to offset the special privilege enjoyed by such companies in competition with power produced from coal, the revenue thus secured going to reduce the general burden of taxation. On thorough consideration, the water powers in all sections of the United States may be found to be ample, so that the same policy could be adopted for the Nation as a whole, using Government coal burned at the mine to supply cheap power to those sections unfavorably located with respect to cheap water power.

The present era of unregulated competition must come to an end in the near future. In its place we will have a National water-power trust, in either private or public control. A water-power expert who is familiar with hydro-electric power conditions throughout the world recently expressed surprise at the extensive development of water power in California and the very limited market. The writer has what is believed to be reliable information to the effect that a 200 000-h.p. plant is now under construction within 200 miles of San Francisco which expects eventually to deliver power in that city at $10 per horse-power-year, or $8 less than can be supplied by its nearest com-

Mr. Lewis.

petitor. The result is inevitable. The water-power trust must purchase this plant, sooner or later, thus adding additional equipment which is not needed at the present time, and the people will have to pay dividends on such "hold-up" prices. Owing to such extravagance, which is forced on the company in control, power rates cannot be greatly reduced to stimulate sales, and the new industries requiring large quantities of cheap power cannot secure a foothold in such State.

Mr. Cory has had wide experience in railway work. He admits the successful handling of reclamation work by the U. S. Reclamation Service, saying that the "Reclamation Service gets more actual work done for a dollar than the Harriman Lines do," in certain cases coming under his observation. He says:

"Such a result is due to the fact that National politics has been eliminated from the Service, but still affects the Harriman Lines somewhat; that the Director of the Service has more authority and freedom in using his individual judgment than even Mr. Lovett of the railroads has, let alone the local presidents; that the head officials, and consequently the general ideas of management of these railroads, have been changed, and the Director of the Service has not; that there is much less internal politics in the Service; and the Service organization and work, as yet at least, has not brought about the sharply drawn lines of cleavage between various departments and the leveling routine of 'common standard' methods, channels of communication, limitations, prerogatives, etc., of the railroad."

The writer has had several years' experience on engineering work for the Harriman system, has served in the U. S. Reclamation Service for about the same length of time, and for the past eight years has had charge of water right and irrigation matters for the State of Oregon. He is thoroughly conversant with the value of system, and of thoroughness in the perfecting of a large organization for a complicated technical work. He is also aware of the instability of the average legislature, and its lack of respect for stability and permanence in organization. Often the person seeking a job has more influence with the State Legislature, through influential members, than a responsible State officer elected by the people and under bond to perform his duty. There is no opportunity in State work for a young person to enter the service as a life occupation. Merit is not, and cannot be, rewarded under such an unstable system with frequent changes. Where work is intrusted to a board, and new officers are elected every few years, a conscientious officer is likely to see years of patient work along some definite policy destroyed by the adoption of some new and antagonistic course of action which the inexperienced majority honestly believes to be the best, at least the best for personal political reasons. Salaries are never equitably adjusted.

State Work.

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With this experience in private and public work, the writer had come to the same conclusion as Mr. Cory with respect to the success of Government irrigation. The criticisms offered by the general public are usually of a minor nature, and little is ever heard of real engineering mistakes. The present difficulty of the Service in colonizing its lands can easily be overcome through co-operation on the part of the States which are most vitally interested. This is the weakest part of the Reclamation Act.

Believing that it is preferable to expand an existing organization which has already made a success of its work, than to create a new and independent organization to work in a complicated field without harmonious relations with the larger organization, the writer is now urging co-operation between the State and Nation in the construction of both irrigation and power projects.

By having a local man representing each State, in active co-operation with the Federal Bureau, many of the objections now urged against the Washington Department will soon be overcome. It will be far easier to overcome these minor objections in administration than to create a new bureau. Stability of organization, and the following out of a fixed policy through many years until the funds advanced are actually returned with interest is essential to development on a large scale. If the States put up one-half the money for such work, closer scrutiny of all expenditures will be had, and funds will not be invested in particular projects for political reasons only.

It would thus appear from the very close relation between irrigation and water power, from the very wide range over which electric power can be transmitted, and from the innumerable legal and practical complications, that it is extremely unlikely that the United States will ever turn over to the individual States her undeveloped water powers; also, it is believed that such action would lead to far more confusion than the policy of co-operation for the immediate development of both power and irrigation possibilities without regard to State lines.

Conclusion.

In closing this discussion on the question of State and National water laws, the writer wishes to urge on all engineers the importance of studying the question from the standpoint of what the laws ought to be, to stimulate investments in works for the utilization of water. The lawyer is paid to protect private rights, and to ascertain what the law is. His experience and training totally unfit him for the important duty of framing constructive water legislation. Engineers, perhaps, are more responsible for the present chaotic condition of water laws than the members of any other profession.

Mr. Lewis.

The present law is very unsatisfactory, from the standpoint of the investor. There must be some Governmental agency to which he can go to ascertain what the law is, how much water in any stream is already vested, the extent of surplus water belonging to the public, and what procedure must be followed in establishing a definite right to such surplus. He must also have assurance that when his money is invested, the State or the Nation will protect him in his right, and not leave him to wage an unequal fight in the Courts, with perhaps a great Commonwealth of the Union or with one of the National bureaus interested in the stream.

If it is proper at this time, and it is within the province and scope of the work of the Society, the writer would like to suggest that, as soon as convenient, the President of the American Society of Civil Engineers be authorized and directed to appoint a commission of engineers to draft a law dealing with interstate and National problems relating to water, and to publish such draft as soon as possible for comment and criticism. At the close of such discussion, perhaps the same commission should be called on to reconsider such proposed law in the light of any criticisms offered, and present the same at some appropriate time for adoption by the Society. Such bill could then be forwarded to Congress and might serve as the foundation for needed constructive water legislation.

A programme of this nature would greatly stimulate research and study along this line, promote discussion, and serve to educate the public as to the necessity for reform in both State and National water laws.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1257

PREVENTION OF MOSQUITO BREEDING.*

By Spencer Miller, M. Am. Soc. C. E.

With Discussion by Messrs. Harold Farnsworth Gray, J. J. Rosenthal, E. P. Felt, Ralph H. Hunt, Robert A. Rutherfurd, Kenneth Allen, W. E. Britton, Thomas H. Means, and Spencer Miller.

In order to promote the health and comfort of the public, mosquito breeding must be prevented as far as practicable in many parts of the United States.

Mosquitoes breed in standing or quiescent water. In the summer of 1910, a pest of common house mosquitoes invaded a city. These mosquitoes bred chiefly in the water standing in the sewer catch-basins during the summer drought. During periods of dry weather, these catch-basins are now regularly oiled, and the mosquito pest has been largely reduced. When these basins are flushed by heavy rain-storms, mosquitoes are not produced for about 10 days. Can a practical type of sewer catch-basin be constructed so that it will not breed mosquitoes?

Sewer manholes provided with a pail suspended just beneath the manhole covers have been found to be extensive breeding places for mosquitoes. Can this pail be eliminated, or if not, can it be made to drain off the water?

Engineers who direct the construction of great railroad fills across salt-marsh areas frequently neglect to provide for the adequate drainage of these marshes. The culverts which are provided are almost invariably placed too high to drain the lands sufficiently to pre-

* Presented at the meeting of December 18th, 1912.

vent the breeding of salt-marsh mosquitoes. The latter lay their eggs in mud, and they are hatched as soon as the mud is well flooded. Six broods of these mosquitoes (within the vicinity of New York) occur each season. Each square foot of salt marsh may breed 5 000 mosquitoes in a single brood. An acre of land contains 4 840 sq. yd.; thus, 1 sq. ft. of salt-marsh breeding area will furnish one mosquito for every square yard in an acre of land. Surely 5 000 mosquitoes to an acre must be regarded as a "pest." A square mile contains 640 acres. A pool on the salt marshes, about 25 ft. square, may supply a pest of mosquitoes for 1 sq. mile of territory; 100 000 people could live on 1 sq. mile of land. Such a pool, therefore, could breed mosquitoes enough to torment 100 000 people for six periods during a season; and, furthermore, as mosquitoes may live two weeks, these 100 000 people may be tormented for one-half the summer season. When so great a pest may proceed from so small a source, is it not the duty of engineers employed in railroad work to give full consideration to drainage facilities for the salt marshes which are being crossed?

In other ways, also, the work of engineers may be directly associated with the prevention of mosquito breeding. It was established in the City of New Orleans that the yellow fever mosquito bred most prolifically in elevated water tanks and water barrels. By the introduction of a central water supply system, these water tanks and water barrels were eliminated, and the mosquito pest was thereby largely reduced.

Cesspools are prolific breeders of the common house mosquito. The introduction of a sewer system and the filling up of these cesspools have been known to reduce largely the mosquito pest.

City engineers can order dirt taken from roads in grading to be dumped in low spots where water stagnates.

The civil engineer has an opportunity to serve his fellow men and do a vast amount of good work in the prevention of mosquito breeding, if he acquires the knowledge now available on their breeding habits.

It is not intended in this paper to review the work which has been accomplished by entomologists during the last few years. A number of pamphlets, devoted exclusively to the subject, have been published by the Department of Agriculture, and are obtainable for the asking.

Reports by State Entomologists are also frequently obtainable. Dr. L. O. Howard, Chief Entomologist of the Department of Agriculture, has published an extremely valuable book on the subject, which is obtainable from publishing houses. The late Dr. John B. Smith, State Entomologist of New Jersey, prepared a most elaborate review on the mosquitoes bred in the State of New Jersey, which, if not out of print, is obtainable by application to Dr. Lipman, Director of the State Experiment Station, New Brunswick, N. J. Dr. Ephraim P. Felt, State Entomologist of New York, has prepared a review on the mosquitoes of New York State, which is intended for free distribution. There are numerous other books on the same subject, many of which are printed abroad.

The writer quotes herewith in full the "Mosquito Brief," prepared, a few years ago, by the American Mosquito Extermination Society, with the aid and endorsement of the members of the Advisory Board of Entomologists. This Brief may be regarded as the A-B-C of the whole problem of mosquito prevention, and is authoritative.

"MOSQUITO BRIEF.

"1.—There are over 100 species of Mosquitoes in the United States.

"2.—Mosquitoes breed only in water. They may breed in any kind of quiet water unstocked with destroying fish.

"3.—Mosquitoes generally require from one to three weeks to develop from eggs to winged insects in warm weather, longer in cold weather. Some female mosquitoes three days old lay eggs, the average is greater. Some species lay as many as three or four hundred eggs at once, some lay them singly. Mosquitoes may live several months (as shown by hibernation and otherwise), but probably few live over a month.

"4.—Mosquitoes do not breed in grass, but rank growths of weeds or grass may conceal small breeding puddles, and form a favorite harboring place for adults. The Pitcher Plant holds sufficient water to breed a rare and small species.

"5.—Different species of mosquitoes have as well defined habits as different kinds of birds, flies, etc. Some are Domestic, some Wild, some Migratory.

"6.—Most Domestic Mosquitoes breed in fresh water, fly short distances and habitually enter houses.

"7.—Most Migratory Mosquitoes breed in salt and brackish marsh areas, fly long distances. They are not conveyers of malaria.

"8.—Rigid tests, both direct and eliminative, have proved that certain species of mosquitoes are the only known natural means of

transmitting malaria and yellow fever. Some other diseases are known to be conveyed by mosquitoes.

"9.—Of the domestic varieties, the dangerous malarial mosquitoes (several species of the genus Anopheles) are among the most generally distributed. They seem never to travel far, only a few hundred yards.

"10.—A most common and dangerous domestic mosquito in the South and the tropics is Stegomyia fasciata, which is the natural conveyer of yellow fever.

"11.—Mosquitoes are known to bite more than once, as can be seen by observation and is proved by the transmission of disease from an infected person to a new subject.

"12.—Mosquitoes are a needless and dangerous pest. Their propagation can be largely prevented by such methods as drainage or filling of wet areas, removal, emptying or screening of water receptacles, spraying standing water with oil where other remedies are impracticable. Attention should be paid to cisterns, house-vases, cesspools, road basins, sewers, watering troughs, roof gutters, old tin cans, holes in trees, marshes, swamps and puddles. As malarial mosquitoes may be bred in clear springs, the edges of such places should be kept clean, and they should be stocked with small fish. The breeding and protection of insectivorous birds such as swallows and martins, should be encouraged. Thorough screening of houses and cisterns is necessary to prevent the spread of malaria or yellow fever. The continued breeding of any kind of mosquitoes with the attendant menace to public health and to the life and comfort of man and beast is therefore the result of ignorance or neglect."

The writer's own observations, which cover the past eleven years, have seemed to establish the fact that the most troublesome varieties of mosquitoes are those bred in man-made water holders. Common house mosquitoes habitually enter houses and are busy all night singing and stinging. When these mosquitoes are ready to lay their eggs, they leave the house for the nearest spot where standing water is to be found. They lay their eggs in rain-water barrels (man-made water holders); in cesspools (made by man); in rain-water gutters which are not kept clean; in fire-pails; in pools formed in excavations, cellars, trenches, etc. These man-made water holders are prolific breeders chiefly because they breed nothing else. Natural pools which hold water for a greater part of the season frequently contain water beetles and other creatures which devour the mosquito larvæ, and thus prevent their development into winged insects.

Another fact established by the writer's observations is that it pays to eliminate breeding places throughout small areas, and that the

greater the area thoroughly controlled the better will be the results. For example, he observed one section where no mosquitoes were seen for two weeks, while people living less than 400 ft. therefrom were suffering severely from the pest. An examination showed that, where the mosquitoes were troublesome, they were breeding extensively in two water barrels.

Another example which came under his observation, was that of a municipality, 2 miles square, which was known to be thoroughly controlled, but was found to suffer considerably from common house mosquitoes which were breeding extensively beyond its borders.

Because of these experiences, he believes that the unit of operation against mosquito breeding should be enlarged. The county becomes a proper sized unit; furthermore, all counties throughout a State should work in unison in order to accomplish the best results. A law passed by the State Legislature of New Jersey during the session of 1912 is printed herewith. It provides for the appointment of commissioners by the Supreme Court Judge of each county throughout the State. It also provides for raising, by taxation, the funds needed to carry on the work of mosquito prevention.

"CHAPTER 104.

"An Act for the establishment of county mosquito extermination commissions and to define their powers and duties.

"BE IT ENACTED *by the Senate and General Assembly of the State of New Jersey:*

"1. In any county of this State it shall be the duty of the justice of the Supreme Court presiding over the courts of said county to appoint six persons, three of whom must be persons who are or have been members or employes of boards of health. A board of commissioners to be known as 'The County Mosquito Extermination Commission,' inserting the name of the county in and for which the commissioners are appointed. The commissioners first appointed under the provisions of this act in any county shall hold office respectively for the term of one, two, and three years, as indicated and fixed in the order of appointment, and all such commissioners, after the first appointment, shall be so appointed for the full term of three years; vacancies in the said commission occurring by resignation or otherwise shall be filled by such justice, and the persons appointed to fill such vacancies shall be appointed for the unexpired term only; such persons so appointed, when duly qualified, constituting such commission and their successors are hereby created a body politic, with

Judge of Supreme Court to appoint commission.

Official name.

Terms.

Vacancies.

power to sue and be sued, to use a common seal and make by-laws;
Expenses paid. the members of any such commission shall serve without compensation, except that the necessary expenses of each commissioner for actual attendance on meetings of said commission shall be allowed and paid.
Oath. No persons employed by the said commission shall be a member thereof; before entering upon the duties of his office each commissioner shall take and subscribe an oath or affirmation before the clerk of the county in and for which he is appointed to faithfully and impartially perform the duties of his office, which oath or affirmation shall be filed with the clerk of the county wherein the commission
Organization. of which he is a member is appointed; every such commission shall annually choose from among its members a president and treasurer, and appoint a clerk or secretary and such other officers and employes as it may deem necessary to carry out the purposes of this act; it may also determine the duties and compensation of such employes,
Office. and make all rules and regulations respecting the same. It shall be the duty of the board of chosen freeholders in each county to provide such commission with a suitable office where its maps, plans, documents, records, and accounts shall be kept, subject to public inspection at such times and under such reasonable regulations as the commission may determine.

Duty of director of experiment station. "2. The director of the State Experiment Station shall be a member *ex-officio* of each commission and shall co-operate with them for the effective carrying out of their plans and work. The said director shall serve without compensation, except that the necessary expenses actually incurred by him in the attendance on meetings of said commissions shall be allowed and paid. He shall furnish the said commissions with such surveys, maps, information, and advice as they may require for the prosecution of their work, or, as in his opinion, will be of advantage in connection therewith.

Powers. "3. Every such commission shall have the power to eliminate all breeding places of mosquitoes within the county wherein it is appointed, and to do and perform all acts and to carry out all plans which in their opinion and judgment may be necessary or proper for the elimination of breeding places of mosquitoes, or which will tend to exterminate mosquitoes within said county.

Annual estimate. "4. Said commission shall, on or before the first day of April in each and every year, file with the director of the State Experiment Station a detailed estimate of the moneys required for the ensuing year, and a plan of the work to be done and the methods to be employed. The said director shall have the power to approve, modify or alter the said estimates, plans and methods, and the estimate, plan and method finally approved by him shall be by him forwarded to the board of chosen freeholders in each county on or before the first day of May following its receipt.

"5. It shall be the duty of the board of chosen freeholders of each county, or other body having control of the finances thereof, to include the amount of money approved by the director of the State Experiment Station, annually in the tax levy; *provided, however,* that in no year shall the amount so raised exceed the amount hereinafter specified, to wit, in counties where the assessed valuations are not more than twenty-five million dollars, a sum not greater than one mill on every dollar of assessed valuations; in counties where the assessed valuations are not more than fifty million dollars a sum not more than one-half of one mill on every dollar of assessed valuations; in counties in which the assessed valuations are in excess of fifty million dollars a sum not more than one-quarter of one mill on every dollar of assessed valuations. Appropriation. Proviso; amount raised annually.

"6. The moneys so raised, or so much thereof as may be required, shall be paid from time to time to the said mosquito commission on the requisition of said commission, duly signed and approved by the president and secretary thereof. Payments.

"7. It shall be the duty of each commission annually, on or before the first day of November in each year, to submit to the director of the State Experiment Station and to the board of chosen freeholders in their respective counties a report setting forth the amount of moneys expended during the previous year, the methods employed, the work accomplished and any other information which in their judgment may seem pertinent. Annual report.

"8. Nothing in this act shall be construed to alter, amend, modify or repeal the provisions of chapter 134 of the laws of 1906, or alter, amend, modify or repeal any act now existing conferring upon State or local boards of health any powers or duties in connection with the extermination of mosquitoes in said State, but shall be construed to be supplementary thereto. Act considered supplementary.

"9. This act shall take effect immediately.

"Approved March 21, 1912."

The Essex County, New Jersey, Mosquito Extermination Commission was organized in March, 1912, and under this law obtained $75 000 from the Board of County Freeholders to carry out the work in the county during 12 months ending May 1st, 1913. There are about 600 000 people in Essex County, and this tax amounts to 12½ cents per inhabitant per annum. The work accomplished during 1912 has been extraordinary; on every side the same story is heard: "We have never known a year when we have seen so few mosquitoes. We might almost say we have hardly seen one in our house this year."

The organization of this Commission is as follows:

1.—Six Commissioners serving without salary.

2.—One Chief Inspector, salary, $1 800 per annum.

3.—One Secretary, salary, $1 200 per annum.

4.—One Assistant Chief Inspector, $1 500 per annum.

5.—Three Deputy Chief Inspectors (who must qualify as engineers), salary, $1 200 per annum.

6.—Forty-five Inspectors at $3 per day.

7.—Thirty to forty Laborers at $2 per day.

8.—Stenographer and Clerk, salary, $600 per annum.

Each inspector patrols a distinct district, making his rounds once in every 10 days. In certain sections of the City of Newark, a day's work for an inspector is from 100 to 300 houses; a day's work for an inspector in the outlying districts is far less. All breeding places which cannot be eliminated at once are recorded on maps made by the Deputy Chief Inspectors.

In closing, the writer wishes to emphasize:

1.—That all standing water does not breed mosquitoes, and great economy can be effected in the cost of drainage and oiling if each pool of standing water is carefully examined for mosquito larvæ before any work is expended thereon.

2.—That we will never be relieved of the work of mosquito prevention. The fight will be continuous, and any slackening of vigilance on the part of the organization will invite the immediate reappearance of the pest.

DISCUSSION

Harold Farnsworth Gray, Jun. Am. Soc. C. E. (by letter).— Within the short space of this paper, the author has compressed considerable information on the subject of mosquito control. It is hoped that the paper will call to the attention of engineers the fact that, in many instances of design and construction, there are details which, if overlooked, may be the cause of considerable annoyance on account of mosquito breeding. Mr. Gray.

The writer gains the impression, on reading this paper, that Mr. Miller's experience with the mosquito problem has been obtained largely in New Jersey and contiguous territory, where it is more a problem of nuisance than of health. Considered on a National scale, however, mosquito control is primarily a problem of health, and is rural rather than urban. In general, it becomes an urban problem only when there is a lax administration of the municipal departments of health and public works.

The author's observations on cesspools as breeding places of mosquitoes have been confirmed by the writer in many cases. A particular illustration of this point came to his attention two summers ago, in a town of about 5 000 population. At that time a sewer system was being constructed, and completion was expected in September. On that assumption, the residents did not keep their cesspools in proper repair, in view of the expected expense of sewer connections in the near future, and a large number of cesspools were in very bad condition. In June a pest of mosquitoes resulted, and it required nearly a month of continuous work to get the situation in hand. Under ordinary conditions, however, cesspools, properly constructed and cared for, do not breed mosquitoes. It is also significant that the Anopheline (malaria-transmitting) mosquitoes do not breed in cesspools, except in rare instances.

Mr. Miller has failed to mention street gutters as sources of mosquitoes. This is a common condition in California, on streets with practically no grade, especially if the gutters are fouled with rubbish. The regular use of a broom would abate the nuisance.

Railroad construction very often diverts or obstructs the natural drainage of the region through which the line passes. The Southern Pacific Railroad through Placer County, in California, furnishes numerous examples. Between Auburn and Roseville on this line the region is intensely malarial, and the railroad contributes a large number of breeding places. The difficulty has been caused by the lack of sufficient culverts, or by culverts improperly placed, and could have been obviated at slight cost at the time of construction. A portion of this line has been greatly improved, partly by ditching and partly by

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filling on the higher side of the embankments. The writer would call to the attention of engineers the fact that it is not the collection of water in considerable volume and depth behind embankments which is responsible for mosquito breeding. This might occur if no culverts at all were placed under a high fill. It is the small, shallow pools which breed the Anophelines, and, to prevent such slight accumulations, water culverts must be placed more carefully than has been the custom.

A matter not mentioned by the author, and probably the most important in engineering interest, is the eradication of mosquitoes and malaria on irrigation projects. In California, this problem is decidedly prevalent, practically no project in the Sacramento-San Joaquin Valley being exempt. The establishment of irrigation has often been marked either by the introduction, or the noticeable increase, of malaria. Irrigation projects are financed and executed as commercial propositions, and anything that affects unfavorably the sale of such lands means a financial loss in two ways, first, the selling value of the land may depreciate, second, the sales may be retarded, tying up capital for a longer period than anticipated. Malaria is a very potent factor in depreciating the value, and retarding the sale, of land. In regions which are potentially malarial, the engineers in charge of irrigation projects should investigate carefully the possibilities of mosquito breeding, and at the time of construction make such provisions as are necessary to prevent the accumulation of stagnant water. The writer hopes that members of this Society who have had to cope with such conditions will add their data to the discussion of this paper.

It may be stated, as a general proposition, that it is the misuse of irrigation water, or defective irrigation systems, and not irrigation in itself, that is responsible for mosquito breeding and malaria on irrigated lands. The writer need not call to the attention of this Society the fact that irrigation unaccompanied by such drainage as is necessary to remove promptly all excess water is bad engineering and agricultural practice, as well as unhealthful. In California, attention to this matter is apparently seldom given. There are a number of cases on record where whole districts have been ruined by irrigation in which there was no provision for drainage, and in certain large areas the ground-water has been raised to an alarming extent. Under such conditions, mosquitoes and malaria become an important factor in retarding commercial and agricultural development.

In the writer's opinion, the solution of the mosquito problem on irrigated projects lies in the following measures:

(1) Improvement, not obstruction, of the natural drainage courses;
(2) Adequate artificial drainage;
(3) Prevention of seepage from ditches, with its resulting collection of water in adjoining low areas;

(4) Proper leveling or surfacing of the land, to eliminate small low areas; Mr. Gray.
(5) Maintenance of canals, ditches, and drains free from obstructions to flow;
(6) Strict regulation of the use of water, to avoid excessive use and waste;
(7) The occasional use of oil or larvicides on isolated collections of stagnant water which are too small or temporary to require drainage.

With the author's selection of the county as a proper unit of mosquito control operations the writer heartily concurs, but with the condition that the field of the county's operations shall not include incorporated cities. The city problem, as far as local breeding is concerned, is best handled by the city health department. The advisability of a special county mosquito extermination commission is questionable. The writer does not deny that in New Jersey, where the mosquito problem is largely due to salt marshes, a special commission may be advantageous; but the average mosquito problem is very directly one of public health, and, therefore, should be a part of the work of the county health office. The writer is convinced that, as a general rule, the appointment of special commissions is an unnecessary multiplication of executive bodies in the county government. It may be objected that the average county health department is inefficient, and that it does not argue well for the success of anti-mosquito measures to trust them to such a department. This may be granted, but, on the other hand, it may be argued that no department of county government, on the average, is remarkable for its intelligence and efficiency, that there is no assurance that a special commission will show greater efficiency than the existing component parts of the county government, and that the increase of executive bodies under such conditions can only result in duplication of functions and still greater inefficiency. Efficiency in government is to be obtained by concentration, not dispersion, of functions and authority.

In California, in spite of a crying need for anti-mosquito work over a very large area, practically nothing has been done by the county governments. The only attempt in this direction, so far as the writer is aware, has been the passage of an ordinance by the Board of Supervisors of Tehama County. This ordinance was constructed by the writer, and was designed as a club in the hands of a certain irrigation company which is trying, with fair success, to eliminate malaria on its project. The county officials have provided no funds for the enforcement of this ordinance, although the economic loss to this county from malaria is certainly not less than $100 000 annually. The expenditure of $5 000 annually by the county for the administration of anti-

Mr. Gray. mosquito work (the cost of drainage to fall on the owner of the land benefited) would make it possible practically to wipe out this economic loss in 5 years, with a net economic saving in this period of about $330 000.

With the author's statement, that "The civil engineer has an opportunity to serve his fellow men and do a vast amount of good work in the prevention of mosquito breeding, if he acquires the knowledge now available on their breeding habits," the writer can agree only within certain limits. He believes that engineers, simply as engineers, are not qualified to undertake in an executive capacity the direction of anti-mosquito measures of magnitude, as such work, for highly successful prosecution, requires marked abilities as an educator and publicist, as well as a unique combination of thorough knowledge as to engineering, parasitology, hygiene, vital statistics, law, and government. The remarkable abilities of Sir Ronald Ross illustrate the point in question. At the same time, engineers can render distinct and valuable service by making certain that the works they design and construct do not become, through mosquito breeding, a menace to the health and comfort of the people.

Mr. Rosenthal. J. J. ROSENTHAL, ASSOC. M. AM. SOC. C. E. (by letter).—The writer is deeply interested in this paper, as he has experienced much unpleasantness from this pest on Corregidor Island, Philippines, a military post about 3 miles square. The houses are screened, but only by using mosquito nets on the beds can one insure any sort of protection from these insects at night.

About four years ago, before active construction was commenced, and when there were fewer inhabitants, there was no trouble from mosquitoes. The rains, producing pools in natural depressions, were as heavy then as now, so that the influx of the pest must have been due to the numerous uncovered wooden water tanks and barrels introduced by the U. S. Engineers and the Quartermaster's Department, wherever work was going on. An effort is now being made to eliminate the insects by destroying all unnecessary water receptacles and building reinforced concrete covered reservoirs with screens over all openings.

The Island of Caballo, about half the area of Corregidor, and only ½ mile south of it, offers a pleasing contrast. At this place, when construction was begun by the U. S. Engineers, all water receptacles were kept covered, and care was taken to prevent the accumulation of any stagnant water, hence the mosquito has not yet found a home there.

In Manila the pest still breeds heavily, due, in part, to the uncleanliness of the natives and to inadequate sewerage and water connections, especially in the localities where the nipa (palm-leaf) and bamboo huts still abound.

Mr. Felt.

E. P. Felt, Esq.* (by letter).—The studies and observations on mosquitoes in various localities in the State by the writer's department, convince him that the engineer may play a very important part in the control of mosquito breeding, because so much of it is purely local. He is thoroughly convinced, for instance, that the incidental pool, roadside or other, is a more important factor in the production of mosquitoes than the larger bodies of water so frequently viewed with distrust by the general public. The engineer has much to say as to methods, in all undertakings where his services are necessary, and it frequently happens that very little or no additional expense is required to eliminate small collections of water, sometimes ignored by members of that Profession.

The writer is particularly interested in Mr. Miller's reference to sewer catch-basins and the possibility of eliminating mosquito breeding in such places. It has occurred to him that, if no better method could be devised, it would be comparatively easy to suspend over the collection of water in the catch-basin, a small container filled with petroleum or other oil and adjusted in such a way as to drip slowly for an extended period. This device, in a modified form, has been used most successfully in certain tropical regions where thorough drainage was impractical. The same idea could be applied to cesspools, and also to cisterns, if the water from the latter was taken from below the surface.

The writer heartily endorses the idea of extending mosquito control over considerable areas. There is more or less migration or drifting of these annoying forms, and, the more extended the work, the better will be the results as a whole. The communities along shore lines, particularly in regions of extensive marshes, are more interested than those farther inland, which may be rarely annoyed by migratory forms, such as the banded salt-marsh mosquito. While it would be desirable to have the unit for mosquito control coincident with county lines, under conditions such as mentioned above, it would seem probable that greater efficacy would be secured by the co-operation of townships or communities suffering from similar conditions. It is difficult, for example, for inland residents to appreciate the desirability of large expenditures for the control of marsh forms practically unknown in their own locality.

Dr. Hunt.

Ralph H. Hunt, M. D.†—The speaker is sure that no one is more fit or better equipped to speak with authority on this subject than Mr. Miller, for in New Jersey he is called the "father" of mosquito extermination and prevention of mosquito breeding, in that he was the first to make of it a live issue and bring to the notice of the

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† President, Essex County Mosquito Extermination Commission.

Dr. Hunt. people the fact that the mosquito could be placed under control and rendered less of a nuisance.

Mosquito work began some years ago, under the efforts of an English Army surgeon, Sir Ronald Ross, who demonstrated that only through the mosquito, in this case a variety of *Anopheles*, could malaria be carried from man to man, and that it was the only known carrier of the infection. Previous to that time it had been supposed that malaria was caused by a miasma or by something in the atmosphere, the idea of the disease being very indefinite until, through the investigations of Italian observers, the organism was discovered in the blood of the human being. Shortly after that Sir Ronald Ross demonstrated conclusively that the mosquito was the carrier of malaria. This was the first thing that brought to the attention of the people the necessity of getting the mosquito pest under control. It is seen, therefore, that in those days it was purely a health problem.

Later, the United States Army surgeons in Havana, Cuba—three of a commission of four sacrificing their lives in the investigation—demonstrated the fact that a certain variety of mosquito, *Stegomyia fasciata*, was the carrier of yellow fever. That was proven beyond a doubt; and two things stand out in scientific medicine and health problems to-day, namely, that if there were no mosquitoes, there would be no malaria and no yellow fever.

The malaria problem would be done away with entirely by the extermination of the variety of *Anopheles* which carries the disease; but, the habits of that mosquito being more difficult to control, it has been less controlled, even where it is a menace, than the yellow fever mosquito in yellow fever countries.

The control of the yellow fever mosquito is comparatively easy, owing to its habits of breeding. It seems that it breeds only in barrels and other small receptacles around houses. One has only to prevent water from standing in these receptacles in order to get rid of the yellow fever mosquito. A complete demonstration of the efficiency of work of that sort has been given in Havana, Cuba, a place formerly pestered with yellow fever but to-day absolutely free from the disease. Yellow fever formerly was a positive menace, and retarded the industrial development of the city; to-day there is no more quarantining of the port, and no more shutting up of business.

We know that the Isthmian Canal could probably never have been completed had not this scientific question been met prior to commencing the mechanical work. In the Canal Zone to-day there is no yellow fever, and mosquito control alone has accomplished this wonderful result.

Referring to the speaker's section of the country, namely, Essex County, New Jersey, which place has been pestered from the time it was first colonized, in early days, to the beginning of the present season

we have reports from Colonial times stating that it was almost impossible to live in certain parts of this section of the country on account of the mosquito pest. The Essex County Mosquito Extermination Commission holds that it has faced a difficult problem and has worked it out to a practical solution for the first time, as far as the speaker knows, in the history of the country. Dr. Hunt.

In previous years local boards of health had attempted to control the mosquito, and through them a desultory movement began. Mr. Miller, who was in this work at its inception, states that it began about eleven years ago, and the speaker assumes that that was about the time when the first practical work was attempted by local boards of health with respect to mosquito control. Up to 1912, it was found that such a method was impractical. The local boards of health could not obtain, or would not spend, the money, and even those that did make an effort to do their share in the extermination of the mosquito, suffered greatly from the inroads of mosquitoes which were bred by their neighbors.

This led to an attempt to formulate a law by which to get control of the whole problem, and be sure that, if there were six little towns or villages in the county, instead of having two do the work and suffer from the lack of work on the part of the other four, some arrangement could be brought about whereby all six municipalities would do enough work to control it. That led to the passing of the present county law, by which, as Mr. Miller has stated, county commissions are established. Each commission is given funds in accordance with the law, rated by the taxable valuations of the county. The commission formulates its plans, states the amount of money which it will need for the season's work, and sets forth the need of the work to be done. This is forwarded to the Director of the State Experiment Station, and his "O. K." makes it mandatory on the board of freeholders to furnish the money on the request of the commission. The commission is appointed by the chief justice of the county. This relieves the situation and takes the question out of politics. The commission serves without pay, and, up to the present time, this system, as it is being tried in Essex County, has been found to be admirably adapted to the problem.

The Essex County Commission has met with much criticism, but in spite of that it has gone ahead with the work. The members have felt that they know the problem, know the mosquito, and know how it breeds; they know how it can be controlled, and they have men willing to put in the time and do the work, and, under the new law, they have the money.

The law was framed so that the appropriation was not available until a certain proportion of the county had been flooded with mosquitoes which bred on the salt marshes, but, after that, the work was

Dr. Hunt. carried out as thoroughly as possible. As a result, both the local papers and the general sentiment of the communities were unanimous in the opinion that they had never known a season when mosquitoes were so few in Essex County as the one just past. Weather conditions may have been favorable for the Commission during 1912, but it is known that wherever mosquitoes have been found heretofore they were found to be breeding during the past summer, but there were enough men to control the situation. It seems almost useless to continue the discussion of the paper, because the speaker's work and that of Mr. Miller naturally have been on parallel lines; his plans are the speaker's plans on this subject, and the results are the same.

To reiterate: There is in Essex County, as Mr. Gray has stated, not only the salt-marsh mosquito, but the house mosquito, the *Culex pipien,* which has proven a greater nuisance than either of the salt marsh varieties. The *Culex cantator* and *Culex solicitans* breed only in salt water. These mosquitoes have been found in Morristown, Summit, and Bernardsville, and also at points from 20 to 25 miles from the nearest salt marsh. It is apparent, therefore, that these varieties are migratory, and are capable of flying 25 miles. They are most prolific, and can be controlled only by ditching the salt marshes. The salt marshes on the Hudson County Meadows and on the Union County Meadows are being ditched thoroughly, and it is hoped and believed that the salt-marsh mosquito can be eliminated from Essex County and from New Jersey, if this work is continued in its present form.

The *Culex pipien* is a small mosquito which breeds in fresh water, and the dirtier the water, the better it likes it. It breeds in brooks, in old tin cans, in pools, and in any place where there is water. It requires a house-to-house inspection for its control. It is a continuous nuisance, and to eliminate it each inspector must visit and revisit every house in his district. Essex County was divided into as many districts as seemed wise, and an inspector was placed over each one and made directly responsible to his chief, and the chief to the Commission.

The above-mentioned mosquitoes are practically the only three varieties which are found in Essex County. We know where they breed; we know how they breed; and we know how to beat them; and, if the money is still forthcoming in future years, we believe it to be inevitable that the mosquito must go.

All will acknowledge the fact that it would be a wonderful thing for New Jersey or for any community to rid itself of mosquitoes. One can hardly estimate just what it means in real estate values alone. It is within the experience of everybody who lives in New Jersey that men who have gone there to occupy property and to make it a permanent home, have suffered from this little pest, the *Culex*

pipien, and the salt-marsh mosquitoes, and have gone away and refused to invest in property in that State. Therefore, in Essex County, it is considered a good investment to devote $75 000 toward the extermination of the mosquito and set it against the depreciation in real estate and the actual discomfort which comes from the pest. Dr. Hunt.

The speaker is a practicing physician, and during 1912 has been somewhat interested in the amount of malaria in the county. He thinks that the mosquito problem is more of a nuisance than a health proposition; not that we do not have malaria in New Jersey, for we do have it, but it is a negligible quantity, and we would not feel justified in spending any such sum of money to get rid of what little malaria there is in the county. In doing away with this nuisance, however, we will destroy the *Anopheles* mosquito, which is the carrier of malaria.

The speaker is very glad that Mr. Miller has written this paper, and feels that everywhere we go we should preach this gospel. If intelligent men could be interested in the subject, another step in the fight would be won. It is hoped that each engineer will do all he can in his community to create an interest in this work and aid it in every possible way.

The mosquito acts as an intermediate host. The malarial organism goes through an entirely different stage in the mosquito from what it does in the body of man. This is also true in yellow fever, as far as is known. It cannot be traced in yellow fever, because the organism has not yet been determined, but in the case of malaria the organism has been traced in all its stages from man to mosquito and from mosquito back to man.

Robert A. Rutherfurd, Esq.—In Connecticut, a number of people interested in the mosquito question, including the speaker, are trying to obtain some legislation, based on the effective New Jersey law, but modified to suit the different political and physical conditions. Mr. Rutherfurd.

The people are bothered to a great extent with salt-marsh mosquitoes which can be eliminated by drainage of the salt marshes. One of the proposed bills places the entire direction and execution of the marsh drainage part in the hands of the Director of the State Agricultural Experiment Station. Another bill deals with the breeding of fresh-water mosquitoes, and empowers boards of health to remove them like any other nuisance.

From all the preliminary missionary work that has been done, it looks as if favorable results would be obtained from the efforts which are being made at Hartford.

The proposed bill putting the marsh drainage work, etc., in the hands of the Director of the State Agricultural Experiment Station calls for an appropriation of $200 000, and gives power to investigate,

Mr. Rutherfurd. inspect, and survey all suspected breeding marsh areas, and also gives him authority to hire engineers, etc., to survey and map such areas, and to estimate and inspect such work as required. The engineers should have a knowledge of mosquito breeding marshes.

Some private anti-mosquito work in several of the small towns has given very good results, particularly the ditching of salt marshes; and, of course, as Mr. Miller states, that has to be maintained from year to year, and kept in proper shape, or the mosquitoes go back and make worse breeding places than ever.

Mr. Allen. KENNETH ALLEN, M. AM. SOC. C. E.—Reference has been made to the breeding of mosquitoes in catch-basins. These basins may be very prolific sources of such pests in our cities, and very little has been done to prevent them from becoming breeding places. When in Washington, in September, 1912, the speaker was interested to see that active preventive measures were being taken to that end, under the direction of Asa E. Phillips, M. Am. Soc. C. E., Superintendent of the Sewer Department. The catch-basins, of which there are more than 5 000, are flushed regularly, once a fortnight, and are given a dose of about a cupful of oil during the breeding season. The cost of each application is about 6 cents per basin. The catch-basins are cleaned once in 4 weeks throughout the year, the flushing being merely to remove the stagnant water.

Cesspools also afford favorable conditions for mosquito breeding, and it will be interesting to note what change may be observed in a city like Baltimore, where 100 000 cesspools are being abandoned on the introduction of sewerage, and where, in certain localities, this insect has been a serious nuisance.

The dissemination of disease by mosquitoes has been suspected for many years, although the conclusive proof is recent. Dr. Howard A. Kelly, of Baltimore, in a paper before the Johns Hopkins Medical Society, mentions some native writings of the Sixth Century, discovered by the Governor of Ceylon about ten years ago, in which insects are mentioned as carriers of disease, and certain kinds of mosquitoes which cause malaria are described.

In 1848 Dr. Josiah C. Nott, of Mobile, Ala., expressed the opinion that yellow fever was transmitted by insects, but it was not until 1886 that the disease was charged to the *Stegomyia fasciata* by a Scotchman then living in Havana—Dr. Carlos J. Finlay; and then some 14 years elapsed before this was demonstrated to be a fact by Dr. Walter Reed, U. S. A., and his colleagues. Referring to this, Dr. Kelly says:

"One thing must strike every reader of Reed's experimental work—that it is based on nothing which was not easily within the scope of scientific investigation at any time within the last 200 years. Their success rested upon inductive reasoning, accuracy of observation, and

perseverance, and thus affords an added illustration of the old lesson that the simple solution is the product of the master mind." Mr. Allen.

W. E. Britton, Esq.* (by letter).—The writer has been much interested in this paper and realizes that the civil engineer has a great opportunity for effective work in mosquito elimination. Especially is this true in regard to construction work, where some of the very worst breeding places have been formed by carelessly intercepting the natural drainage in the building of railroads and highways across marsh lands. Mr. Britton.

The subject of mosquito control naturally falls into two subdivisions: (1), Salt-marsh or migratory species; and (2), Malarial, rain-barrel, and other local species.

The control of the first species requires a large initial expenditure for drainage. As salt-marsh mosquitoes migrate, often flying many miles, not only the area where they breed, but all the adjacent towns are affected. Local effort, therefore, is not satisfactory unless universal co-operation is established. This is difficult. It seems to be a wise provision, therefore, that such work be managed by the State, as is done in New Jersey. After the drains have been cut, a small sum must be expended annually for maintenance, and the writer believes that this may be safely imposed on the towns, possibly with a provision that the State may do the work in case the towns fail. It must be done some way.

The second subdivision deals more particularly with a local nuisance, and more properly belongs to the province of the health officer. If the laws are insufficient, more authority should be given to the health officer and more funds provided in order to enable him to cope with the situation.

The control of the rain-barrel and malarial and other non-migratory species may best be effected by frequent inspections of premises, at the expense of the town or city, and by orders issued by local authorities, the cost of the treatment to be borne either by the property owner, or by the municipality or town. State supervision or interference is hardly necessary.

Whether or not county mosquito commissions are desirable will depend on conditions existing in the different States. In Connecticut, for instance, there are county public health associations composed of the town, city, and borough health officers and a few citizens prominently interested in matters of public health. Such organizations at their meetings discuss these problems and adopt general policies; and each individual health officer will solve his local difficulties with a stronger courage if he knows that the organization is back of his efforts.

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Mr. Britton. In mosquito elimination, as in all other questions of great public import, however, a strong public sentiment must be obtained. Thinking people must be awakened. Often laws will help to create that sentiment, but they are of little value unless supported by it. A great educational campaign is needed, and every legitimate means should be adopted to an end which will improve the health, efficiency, comfort, and, ultimately, the financial condition of the people.

Mr. Means. Thomas H. Means, M. Am. Soc. C. E. (by letter).—This is a timely paper and of great value. We have too long considered the mosquito a sort of necessary evil, and it is of great importance to show that it can be prevented, generally at no large expense. The writer well remembers, during boyhood, the old rain barrel, at the corner of the house, filled with wigglers. No one then connected the wiggler with the pest of mosquitoes which bothered them all summer, and, of course, no one had any idea that the chills and fever were in any way connected with the mosquitoes.

In irrigated districts, mosquitoes rapidly become a great nuisance, unless special precautions are taken to eliminate their breeding places. Very little systematic work seems to have been attempted in the matter of mosquito prevention in the West. Few engineers seem to realize how easy it is to lessen the number of mosquitoes to such a degree that they cease to be a pest. It is to be hoped that Mr. Miller's paper will serve to increase the interest in the matter.

Within the last two years the writer has been engaged on a small irrigation project in the Sacramento Valley. Here, as everywhere else, mosquitoes are a nuisance, but here the matter is much complicated by the presence of the species which distributes malaria. The problem of preventing or lessening malaria is vital to the success of the project. During the summer of 1911, the first year of the writer's connection with the enterprise, no attempt was made to control the matter, largely because no one appreciated its importance. Experience during that season, however, showed the absolute necessity of subduing the malaria mosquito. Plans were made and work was commenced in the season of 1912.

The problem was attacked from all sides. An attempt was made to prevent the breeding of mosquitoes, to isolate the human carrier of the disease, and to encourage the people of the community to take good care of their general health by sleeping under screens, by using good drinking water, and by taking quinine as a preventive. There was no measure of the amount of malaria in the community previous to 1912, except the records kept by the school teachers. These showed that for the previous three years about 80% of the children of school age were sick with the disease. In the fall of 1912, after the one season's work, the percentage of children sick with malaria was reduced to about 15. During this year there had been a great influx of

new people, and, had there been no change in the conditions, a much larger percentage of sickness might have been expected, for the disease is always worse in the case of new comers. The writer feels that he has reason to be pleased with the results. Mr. Means.

Whether or not the season was favorable, no one can say; malaria in the country generally seemed to be about as usual in 1912. Very few farmers have much faith in the mosquito theory of malaria, and it was hoped, at least, to establish confidence in the method of handling the matter. There was very little co-operation during the year from the farmers, on account of their scepticism, but it is found that now nearly all of them are satisfied that malaria prevention is possible, that mosquitoes have something to do with its spread, and next year they will give their hearty help, and the work should be more successful.

In laying the plans, attention was directed to drainage, as the first and most important matter. Open drains were made, in order to prevent surface water from standing. Where drainage was not possible, oil was used to prevent mosquito breeding. Two men, equipped with knapsack sprays, made regular trips over the country and sprayed every place where mosquitoes could breed. These trips were made at least every two weeks in the warm weather. The territory covered was about 10 000 acres. During the season, 1 400 gal. of oil were used. The total cost of the work was about $3 500. As there were about 1 200 people in the district patrolled, the cost, if pro rated, would have been about $3 per person, which is trifling when the results are considered. The relief from mosquito bites alone would have been well worth the cost.

It has been stated that mosquitoes travel only short distances. This may be true in an unsettled country, but, where there are many people or animals, they will travel great distances. In the evening every traveler will be followed by a swarm of the insects, and every cow coming from pasture will bring them with her.

It is of importance, therefore, to exercise care in preventing the breeding of mosquitoes, even in remote spots.

Malaria mosquitoes seem to prefer the quieter, warmer spots of water. They do not seem to breed in ditches or streams, but every little puddle will produce great numbers. They are a little slower in development than the common varieties, and do not become abundant until the latter part of the season.

As the result of the season's work, the writer would suggest the following for a community troubled with malaria:

1. Drain all surface water.

2. If drainage is not possible, use oil. The cheaper grades of oil from the refineries, such as stove oil, mixed with crude oil, will an-

Mr. Means. swer very satisfactorily. Use a fine spray, and distribute enough oil to form a thin, but visible, film on the surface of the water.

3. Keep the person, known to have malarial parasites in the blood, isolated if possible. Mosquitoes do not generate malaria, but must get it from the blood of a human being. Insist that all malarial persons sleep under screens. Have every such person undergo a thorough treatment under a good doctor.

4. Encourage the use of good water. Deep wells are generally best. Malaria cannot be transmitted through water, but every condition which tends to run the system down will make a person less resistant to the disease.

5. Screen all houses, and, as a special precaution, screen all sleeping places. Keep indoors as much as possible after night. The malarial mosquito is a nocturnal insect.

6. Take quinine during the season when malaria is prevalent. A doctor's advice should be asked. Take a little quinine, no matter what he advises.

Mr. Miller. Spencer Miller, M. Am. Soc. C. E. (by letter).—The writer is much gratified with the reception accorded his paper and the thoughtful discussion it has brought forth. Mr. Gray intimates that the writer's experience with the mosquito problem has been largely obtained in territory where mosquitoes are more of a nuisance than a menace to health. Are not all kinds of mosquitoes a menace to health? Nature provides a period of sleep to maintain health. Mosquitoes which disturb that sleep surely affect one's health; and the greater their number the greater the menace. Recreation in the open air and sunshine maintains health; therefore, a pest of mosquitoes of any variety that drives one indoors is a menace to health.

There are many times more non-malarial than malarial mosquitoes breeding in the State of New Jersey. The malarial mosquito may breed almost anywhere, even in salt marshes, and any campaigning against it must, of necessity, call for the inspection of all quiescent waters. The cost of inspection is the largest part of the cost of extermination. Seeking for malarial mosquito larvæ will lead the inspector to the larvæ of non-malarial mosquitoes. The destruction of non-malarial larvæ adds but a trifle to the expense of a campaign against malaria, and, in the interest of health, such a campaign should be directed against every kind of mosquito.

Mr. Gray states that the problem "becomes an urban problem only when there is a lax administration of the municipal departments of health and public works." In the City of New York, the Department of Health is fully alive to the importance of mosquito prevention. It is lax only because hampered by lack of funds. A large portion of the Borough of Brooklyn is pest-ridden with salt-marsh mosquitoes which

breed in about 9 000 acres of marshes in Jamaica Bay within the limits of the Borough. The Borough authorities have not $10 000 to expend for this purpose at present. It will cost about $60 000 to ditch these marshes alone, and it is estimated that an annual expenditure of about $150 000 per year will be required to relieve properly the Borough of Brooklyn from the mosquito pest. This sum would cost the people residing there less than 10 cents per annum per inhabitant. While salt-marsh mosquitoes probably furnish 75% of the pest in Brooklyn, there are thousands of fresh-water breeding places where the common house mosquitoes breed. These are a greater nuisance individually than the marsh mosquitoes, because they get indoors and sing and bite all night. Mr. Miller.

Mosquito breeding in Prospect Park, Brooklyn, has been known and talked of publicly for the last ten years, yet at the present time the City health authorities have not the funds to prevent it. Citizens of Brooklyn have formed an association and are raising a fund of $60 000 by subscription to start the work.

The writer became interested in forming county mosquito extermination commissions because, after ten years of endeavor, he failed to obtain the co-operation of adjoining communities to do anti-mosquito work effectively.

Mr. Gray concurs in the "selection of the county as a proper unit of mosquito control operations * * * with the condition that the field of the county's operations shall not include incorporated cities."

The Board of Health of Newark, N. J., for five or six years, expended about $5 000 to $7 000 annually. This was about one-sixth of the sum required to do the work effectively within the city limits. After the work was taken over by the County Commission, in the summer of 1912, more than 9 000 breeding places were discovered in the city, entirely outside of salt-marsh areas. Newark has a well-organized and highly intelligent Board of Health, but it was limited financially. Breeding was stopped to a certain extent, but not enough to satisfy the citizens, as will appear from the following editorial comments in Newark papers:

The *Newark Evening Star* on the 15th of July, 1912, said:

"In no previous summer season were Newark and its suburbs so free from the pest as during the present season. Last year in many localities the infliction was unendurable, and the change is most grateful to a large population. The money appropriated for this blessed work is the best public investment ever made by the county. It is being richly repaid in benefits."

On August 17th, the same paper stated: "The mosquito extermination act has been amply justified by results."

Mr. Miller.

The *Newark News*, on July 13th, 1912, said:

"After just two months of activity, backed by an adequate appropriation, it has been demonstrated that the work of the Essex County Mosquito Extermination Commission is accomplishing definite results. * * * After this comparatively short time, results are visible in the lessened number of the pests that find their way into houses and within the confines of screened porches."

On July 24th the *Newark Evening News* stated:

"Sh-h-h!

"Have you noticed it, or are we dreaming! It seems almost too good to be true.

"Has the wily mosquito left us?"

The writer feels that it is necessary to include incorporated cities in connection with county work. A county tax is paid by the taxpayers of the cities as well as the villages. A county commission should expend the money collected by taxation for the benefit of the people who pay it.

Mr. Gray is convinced that "the appointment of special commissions is an unnecessary multiplication of executive bodies in the county government"; and states, further, that "no department of county government, on the average, is remarkable for its intelligence and efficiency"; and "that there is no assurance that a special commission will show greater efficiency than the existing component parts of the county government."

The assurance offered in the New Jersey law, included in the paper, that these commissions shall be intelligent and efficient, is that they shall be appointed by the Supreme Court Judge of each county and shall serve without salary. Mr. Gray states that, in California, there is a crying need of anti-mosquito work, and that practically nothing has been done by the county governments.

He states further: "the county officials have provided no funds * * * although the economic loss to this county [Tehama] from malaria is certainly not less than $100 000 annually." No better argument can possibly be put forth for the necessity of the appointment of specially selected men for carrying out a definite public work.

Mr. Gray says that "highly successful prosecution, requires marked abilities as an educator and publicist, as well as a unique combination of thorough knowledge as to engineering, parasitology, hygiene, vital statistics, law, and government."

The Essex County Mosquito Extermination Commission is composed of: Two doctors, one civil engineer, one retired quartermaster-general, one successful business man, and one grain merchant, all having proven by previous service that they were public-spirited citizens. This Commission has been successful.

Mr. Rosenthal refers to the mosquito pest in the Philippines, which followed the introduction of numerous uncovered wooden water tanks and barrels, "man-made water holders." Mr. Miller.

Mr. Felt speaks with authority when he refers to the incidental pool, roadside or otherwise, as being more prolific in the production of mosquitoes than the larger bodies of water so frequently viewed with distrust by the general public. His suggestion to suspend a small container filled with oil and adjusted so as to drip slowly into a catch-basin has been tried in Newark, N. J. The plan proved effective only when the catch-basins were kept clean. After some days they become partly filled with dirt, and on this the oil may drip and not reach the water; for that reason the plan was abandoned. The catch-basin covers are now removed, and the oil is sprayed all over the surface of the water. What is wanted is a mosquito-proof catch-basin which needs no inspection or oil.

Dr. Hunt views the subject largely from a medical standpoint. The Supreme Court Judge of Essex County was fortunate in having found a man of Dr. Hunt's ability and zeal to serve as a commissioner. Where county commissions are appointed by Supreme Court Judges, the latter would do well to select one or more progressive medical men to serve on the commission, because such men can usually be depended on to be intelligent and unselfish in their desire to relieve the people from the pest.

Mr. Rutherfurd refers to the problem of Connecticut and to a Bill placing the marsh-drainage work and $200 000 in funds in the hands of the Director of the State Agricultural Station. The conditions in Connecticut are peculiar, fully 75% of the population being affected by a mosquito pest breeding along the borders of the State. There is hardly a county which will not be benefited by the elimination of the salt-marsh pest; it is a State nuisance, and a good beginning will be made by the passage of the Bill.

The writer endorses Mr. Britton's suggestion, in which he states that "a great educational campaign is needed." During the season of 1912 the Essex County Mosquito Extermination Commission issued 50 000 cards to be hung in houses. These cards illustrate how mosquitoes breed, and give methods for the prevention of breeding.

The real value of an educational campaign, however, is in moulding public opinion to uphold organized effort for anti-mosquito work. One error frequently made is to expect that people when they know how the mosquito pest can be eliminated, will look after their own premises individually. Experience during ten years has taught the writer that less than 5% of the population will do anything of the sort. No better illustration of that can be found than the fact that the inspectors employed by the Essex County Mosquito Extermination Commission found mosquitoes breeding in the rear of the writer's stables.

Mr. Miller.

In reference to the cost of anti-mosquito work, the following data may serve as a rough basis for making estimates:

1. Port Said, with 50 000 population and compactly built, controls its mosquito pest for 12 cents per inhabitant per annum. Ismailia, an open village, with 10 000 population, expends 50 cents per inhabitant per annum.

2. Klang and Port Swettenham, 5 miles apart, have a combined population of 3 000 and spend $4.72 per inhabitant.

3. Mr. Means, in his discussion, refers to anti-mosquito work in the Sacramento Valley, Cal., in a territory of about 10 000 acres and 1 200 people at $3 per inhabitant per annum. This, however, is only 36 cents per acre per annum.

4. The estimated cost for the Borough of Brooklyn (Kings County), with a population of 1 700 000, is 10 cents per inhabitant per annum.

5. In South Orange, N. J., a suburban community of 5 000 population, the cost of effective work is about 30 cents per inhabitant per annum, or an average of $1.50 per acre per annum.

6. The estimated cost of keeping New Jersey reasonably free from the pest is 29 cents per inhabitant per annum, or an average of 13 cents per acre. This sum is provided for under the New Jersey law quoted in the paper.

7. In Panama, the entire sanitation of the Canal Zone costs approximately $2.43 per inhabitant per annum, and the cost per acre is $1.14. (This is about 1% of the cost of the Canal.) The breeding season is twice as long as that in the latitude of New York or Washington, and probably only one-half of the sum expended was for anti-mosquito work. In order to compare the Panama Canal Zone with New Jersey, these costs must be divided by two, to separate the mosquito work from other sanitary work; and by two again, because the breeding season is twice as long in Panama; and again by two, to compensate for the vast difference in rainfall. The result is 31 cents per inhabitant, or 14 cents per acre per annum.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1258.

THE THEOREM OF THREE MOMENTS.

By J. P. J. Williams, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Milo S. Ketchum and J. P. J. Williams.

The design of continuous beams, plate-girder draw spans, and swing-bridge trusses is based on the values of reactions and bending moments found by the use of a restricted form of the theorem of three moments, neglecting shear deflection. Theoretically, this usual restricted form applies only to beams built to conform accurately to the supports, which are assumed to be perfectly rigid and without possible settlement, and to beams with constant moments of inertia, and straight over the intermediate support. In practice, these conditions are never completely fulfilled. It is desirable, therefore, to determine the approximate value of the error introduced in such common practice. A complete general form of the theorem of three moments will be derived, and its application to a plate-girder draw span with variable moments of inertia will be made, in order to find the percentage of error introduced by the use of the usual formula. The theoretical maximum limit of the error thus introduced for such a typical case will be shown to be about 16.8% on the side of danger. Several fundamental and general relations for continuous beams will be given as the basis of the derivation, and also a direct derivation of the usual restricted form of the theorem. The subject matter will be divided into sections as follows:

Section 1.—The Continuous Beam.—Definition and Use of the Theorem of Three Moments.

Section 2.—Methods of Derivation.

Section 3.—General Deflection Equations for Curved Beams and Arches.
Section 4.—Application to Fixed-Ended Arches.
Section 5.—Application to Simple Beams.
Section 6.—Fundamental General Relation for Continuous Beams.
Section 7.—General Values of Bending Moments, Shears, and Reactions, for Any Span of a Continuous Beam.
Section 8.—Derivation of the Usual Restricted Form of the Theorem of Three Moments.
Section 9.—Effect of Settlement of Supports.
Section 10.—Derivation of the Complete General Form of the Theorem of Three Moments.
Section 11.—Application to Plate-Girder Draw Span with Variable Moment of Inertia.

Section 1.—The Continuous Beam.—Definition and Use of the Theorem of Three Moments.

The continuous beam resting on n supports, at which positive or negative reactions are developed, presents a problem in which the reactions are statically indeterminate. As n unknown reactions are to be found, and only two fundamental equations of static equilibrium are available, the solution of the problem requires $n - 2$ additional equations. The theorem of three moments, which was developed in its original form by Clapeyron in 1857, makes it possible to write the $n - 2$ equations required, and thus solve the problem. It is based on the relation between the elastic distortion in adjacent spans caused by the bending moments in those spans.

Definition of Theorem.—The theorem of three moments is expressed as an equation giving the algebraic relation which exists between the bending moments at any three consecutive supports of a continuous beam and the loading on the two included spans.

This algebraic relation is thus seen to be independent of the loading on the other spans outside of the two adjacent spans considered. It is directly affected by any movement or settlement of the supports, and the general form contains terms which give the effect of such settlement or of nonconformity to supports before loading. The application of the theorem equation to the $n - 2$ pairs of adjacent spans of a continuous beam with n supports, will give $n - 2$ simultaneous equations in terms of the bending moments at the supports and the load-

ing. The two end bending moments being known, there are only $n - 2$ unknown bending moments, which, therefore, can be found at once either by algebraic solution or by the use of determinants. These bending moments at the supports being known, it is possible to find the reactions, also the shear and the bending moment for any span.

Section 2.—Methods of Derivation.

The theorem of three moments may be derived in either of two ways: First, the general bending-moment equation, $M = E\ I\ \frac{d^2y}{dx^2}$, found by the common theory of flexure, can be used to find the slope, $\frac{dy}{dx}$, at the intermediate support, and the values as found from the equations for the two adjacent spans equated when the beam is straight over the intermediate support. Second, a more general fundamental relation between the end deflections for the two adjacent spans, when the beam is not straight, can be derived from the fact that the angle between the tangents at the intermediate support will remain constant, and this relation may be used in the derivation.

The first method is usually given in textbooks, and results only in the restricted form of the theorem. The second method is used by F. E. Turneaure,* Assoc. M. Am. Soc. C. E., for a straight beam with constant moment of inertia, also by W. H. Burr,† M. Am. Soc. C. E., for the perfectly general case of any curved beam. This second method will be applied here, both for the derivation of the restricted form and for the general form of the theorem. It is based directly on a general formula giving the relative vertical deflection, D_v, of any point due to bending, in terms of the bending moments along the beam. This general deflection formula will now be derived.

Section 3.—General Deflection Equations for Curved Beams and Arches.

Let Fig. 1 represent a perfectly general case of a fixed-ended arch subjected to elastic bending stress throughout its length. By assuming a very short length, $N_1\ N = n$, the value of the bending moment, M, can be considered as practically constant throughout such length and equal to its value at, say, N; also, the summations and co-

* "Modern Framed Structures," Vol. II.

† "Elasticity and Resistance of Materials."

ordinates can be taken from N_1 or N, indiscriminately. The differential amount of the deflection of any other point, O, can then be found by considering the movement of the chord, NO, due to the elastic bending in the short length, $N_1 N = n$. For the total deflection caused by elastic bending in the whole length, $N_1 O$ or NO, the summation of these differential deflections can be made. In this manner the general equations for change in angle, and for horizontal and vertical components of total deflection due to elastic bending, can now be derived.

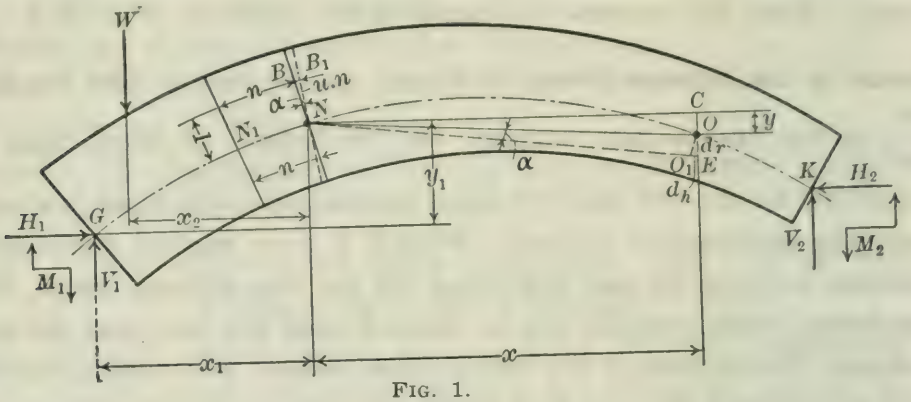

FIG. 1.

Notation.—The notation is indicated by Fig. 1, and also:

n = any small length, $N_1 N$, along neutral axis (theoretically, a differential length; practically, often some convenient length).

M = the general value of the bending moment at any given point, N, considered as an average value for the distance, n, namely, $M = M_1 + V_1 x_1 - H_1 y_1 - \Sigma W x_2$.

α = a small angle in radians between the plane normal section, $N B$, before bending, and its position, $N B_1$, after bending has occurred in the distance, n. The slight effect of variation in distance, $N_1 N$, when the normal planes at N_1 and at N are not parallel, is negligible.

$A = \sum_{O}^{N} \alpha$ = the total angular change = the change in angle between the end tangents at N and O.

d_h and D_h = the differential and total horizontal components of deflection of O, referred to a fixed tangent at N.

d_v and D_v = the corresponding vertical components of deflection of O.

u = the rate of strain at a unit distance from the neutral axis.

f_1 = the intensity of stress at a unit distance from the neutral axis.

E = the coefficient of elasticity.

I = the moment of inertia of the normal section, $N\ B$ = the average for all normal sections in length, n.

Drawing the chord, NO, N is taken as the instantaneous center for the revolution of the point, O, due to bending in the length, n, only, and since no bending occurs yet in the length, NO, the chord remains at a constant angle with the normal plane, NB, and revolves with that plane through the angle, α. OO_1 then represents the deflection of O due to bending in the length, n, and d_h and d_v, the two components of such deflection, are $O_1\ E$ and $O\ E$, respectively.

Now, by the definition of the rate of strain, u:

$$B\ B_1 = un;$$

and, by the common theory of flexure,

$$M = f_1 I = u\ EI. \quad \text{Therefore, } u = \frac{M}{EI} \ldots\ldots\ldots\ldots (1)$$

From the triangle, NBB_1, as α is small:

$$\alpha = \text{tan. } \alpha = un \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Therefore, from Equation (1):

$$\alpha = \frac{Mn}{EI} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Then the total summation of these angular changes in length, ON, would be:

$$A = \sum_{O}^{N} \alpha = \sum_{O}^{N} \frac{Mn}{EI} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Considering the similar triangles, $OO_1\ E$ and OCN:

$$\frac{O_1E}{OO_1} = \frac{OC}{ON}; \text{ therefore, } d_h = O_1E = \frac{OO_1\ OC}{ON} = \frac{OO_1}{ON}\, y.$$

$$\text{Also, } \frac{OE}{OO_1} = \frac{NC}{ON}; \text{ therefore, } d_v = OE = \frac{OO_1\ NC}{ON} = \frac{OO_1}{ON}\, x.$$

But $\frac{OO_1}{ON} = \text{tan. } \alpha = \alpha = \frac{Mn}{EI}$, from Equation (3), which, when substituted in the above values for d_h and d_v, will give:

$$d_h = \frac{Mn}{EI}\, y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

$$d_v = \frac{Mn}{EI}\, x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The total deflections, D_h and D_v, of the point, O, due to bending in the length, ON, therefore, will be the summation of these elementary deflections, giving:

$$D_h = \sum_O^N \frac{Mn}{EI} y \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$D_v = \sum_O^N \frac{Mn}{EI} x \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

It should be clearly understood that the values obtained by making the summations indicated in Equations (4), (7), and (8) will give the resulting effects of bending in the length, ON or ON_1, only. If bending occurs outside the portion, ON or ON_1, considered, it causes a deflection of both O and N or N_1, and affects their relative deflection with respect to each other. Therefore, Equations (7) and (8) give the relative horizontal and vertical deflections of O, with respect to N or N_1, only when the bending does not change the position of the original normal plane of reference at N or N_1. As a tangent to the neutral axis at N or N_1 is perpendicular to such normal plane, it follows that both tangent and normal plane at the end, N or N_1, of the section considered must remain fixed if such equations are to give relative total deflections.

The origin for the co-ordinates, x and y, is at O, the point for which the deflection is being found. It is evident that this is really a moving point, as it is being deflected, but the relative amount of such deflection is insignificant in comparison with the values of x and y.

Section 4.—Application to Fixed-Ended Arches.

Perhaps the most important application of General Equations (4), (7), and (8), is to the case of the fixed-ended concrete or reinforced concrete arch, which is statically indeterminate to the third degree, there being six unknowns and only three fundamental equations of equilibrium, $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$. Such an arch is assumed to have fixed end tangents and to sustain no relative deflections, either horizontal or vertical, between the ends, that is, the abutments are assumed to be absolutely rigid at G and K, Fig. 1. Such assumptions would make the values of Equations (4), (7), and (8), each equal to zero, when the summations are made for the total length.

G to K, giving the following three fundamental equations for elastic equilibrium, which make the problem determinate:

From Equation (4), $$\sum_G^K \frac{Mn}{EI} = 0 \ldots\ldots\ldots (9)$$

From Equation (7), $$\sum_G^K \frac{Mn}{EI} y = 0 \ldots\ldots\ldots (10)$$

From Equation (8), $$\sum_G^K \frac{Mn}{EI} x = 0 \ldots\ldots\ldots (11)$$

It should be noted that these equations are based on deflections due to bending only, and do not account for the effect of direct thrust, shear, or temperature changes.

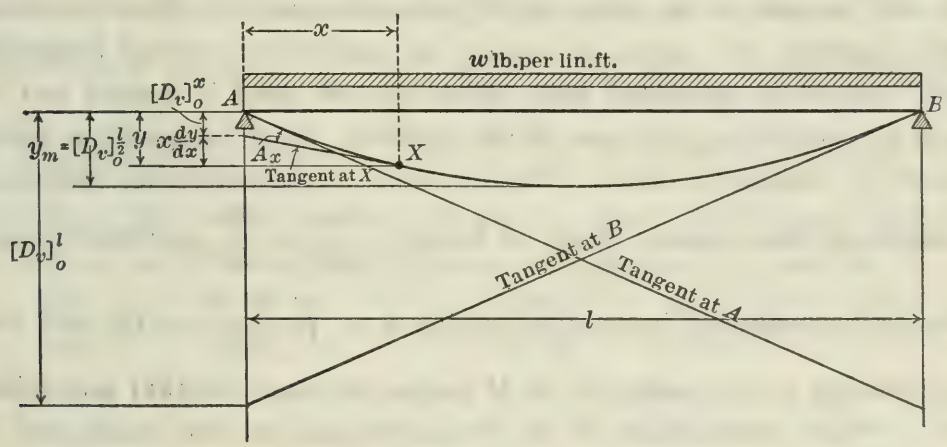

FIG. 2.

SECTION 5.—APPLICATION TO SIMPLE BEAMS.

In order to illustrate the use of Equation (8), giving the relative vertical deflection, D_v, before applying it to the case of the continuous beam, consider the simple beam uniformly loaded, as in Fig. 2. E and I will be assumed as constant and $n = dx$. Then Equation (8) becomes $D_v = \frac{1}{EI}\int_0^x Mx\,dx$, in general for the length, AX. The general value of M, for uniform load, w, per foot, is:

$$M = \frac{w}{2}\, x\,(l - x).$$

Therefore, $$[D_v]^x_0 = \frac{w}{2\,EI}\int_0^x (l - x)\,x^2\,dx = \frac{w}{2\,EI}\left[\frac{lx^3}{3} - \frac{x^4}{4}\right] \ldots (12)$$

If the integration is made for the half length, the maximum deflection, y_m, is obtained:

$$[D_v]_0^{\frac{l}{2}} = y_m = \frac{w}{2\,EI}\left[\frac{l^4}{24} - \frac{l^4}{64}\right]$$

or,
$$y_m = \frac{5\,wl^4}{384\,EI} \qquad (13)$$

If the integration is made for the whole length:

$$[D_v]_0^{l} = \frac{w}{2\,EI}\left[\frac{l^4}{3} - \frac{l^4}{4}\right] = \frac{wl^4}{24\,EI} \qquad (14)$$

It is thus evident that the expression for D_v, when integrated for the total length, l, of a simple span, will not give zero (the relative deflection of the ends), but the value of the vertical movement of the end tangent at the other end, B, below the end, A. That this General Equation (8) cannot be applied to cases where vertical deflection with respect to supported ends which are not fixed is desired can be seen by considering the form of the equation. Such an equation would result by making a double integration of the fundamental deflection equation of the common theory of flexure, $\frac{d^2y}{dx^2} = \frac{M}{EI}$, omitting entirely the first constant of integration, giving $y = \int \frac{Mx\,dx}{EI} = D_v$, and also introducing x as a multiplier of M instead of some constant multiplied by x, which would result if the proper integration were made and M contained terms in x.

The second constant of integration which would now be introduced in either case is always zero, as $y = 0$ when $x = 0$. The first constant of integration is really the value of the slope, $\frac{dy}{dx}$, of the end tangent at the origin, A.

From Equation (14) it is seen that the end tangent at B has a slope, $\frac{wl^4}{24\,EI} \div l = \frac{wl^3}{24\,EI}$. This value can be checked by the ordinary equations of deflection for such a beam, found by integrating the fundamental equation, $\frac{d^2y}{dx^2} = \frac{M}{EI}$, and will be the first constant of integration. It can also be checked by using Equation (4) for change in angle, A, because, for small angles, tan. $A = A$. In general, when $n = dx$ and $E\,I$ is constant, Equation (4) gives:

$$A_x = \int_0^x \frac{M dx}{EI} = \frac{w}{2\,EI} \int_0^x (l - x)\, x dx = \frac{w}{2\,EI}\left[\frac{lx^2}{2} - \frac{x^3}{3}\right] \ldots\ldots (15)$$

The slope of the end tangent at B with respect to the horizontal tangent at the center is then found by integrating from 0 to $\frac{l}{2}$, that is let $x = \frac{l}{2}$ in Equation (15),

$$A = \frac{dy}{dx} = \frac{w}{2\,EI}\left[\frac{l^3}{8} - \frac{l^3}{24}\right] = \frac{wl^3}{24\,EI} \ldots\ldots\ldots (16)$$

The total deflection, y, at any point, x, is not given by the D_v Equation (12), but can be found by adding to $\left[D_v\right]_0^x$ the quantity obtained by multiplying the slope at X by x, as is seen in Fig. 2. The value of the slope, $\frac{dy}{dx}$, at X can be found either by the integration of the fundamental equation, $\frac{d^2 y}{dx^2} = \frac{M}{EI}$, or by using Equation (15) for A_x. The latter can be shown to check the former method as follows, as A_x is the angle between the tangents at A and at X:

$$\frac{dy}{dx} \text{ at } X = \text{slope at } A - A_x = \frac{wl^3}{24\,EI} - \frac{w}{2\,EI}\left(\frac{lx^2}{2} - \frac{x^3}{3}\right)$$

$$= -\frac{w}{2\,EI}\left[\frac{lx^2}{2} - \frac{x^3}{3} - \frac{l^3}{12}\right] \ldots\ldots\ldots\ldots (17)$$

Equation (17) checks the value obtained by general integration, but with opposite sign.

The total deflection, y (using Equations (12) and (17)), therefore, is:

$$y = \left[D_v\right]_0^x + x\left(\frac{dy}{dx} \text{ at } X\right) = \frac{w}{2\,EI}\left[\frac{lx^3}{3} - \frac{x^4}{4} - \frac{lx^3}{2} + \frac{x^4}{3} + \frac{l^3 x}{12}\right]$$

Therefore, $y = -\frac{w}{24\,EI}\left[2\,lx^3 - x^4 - l^3 x\right] \ldots\ldots\ldots\ldots\ldots (18)$

Equation (18) checks the value obtained by general integration, but with opposite sign.

It should be explained that the difference in sign is due to the fact that the D_v Equation gives positive values for deflection when the

positive bending moment occurs, while, in the notation for deflection by the general integration of $\frac{d^2 y}{dx^2} = \frac{M}{EI}$, the deflection is downward or negative for the positive bending moment.

It is also important to observe that the origin for x in the D_v Equation is at the deflecting point, the result being positive if that point has deflected upward with respect to the tangent at the other end of the section considered. No constants of integration are introduced in the foregoing, as they are zero.

Whenever concentrated loads occur, it is necessary to make separate integrations for each segment of the beam, because the law of variation of the bending moment with x will change at each load.

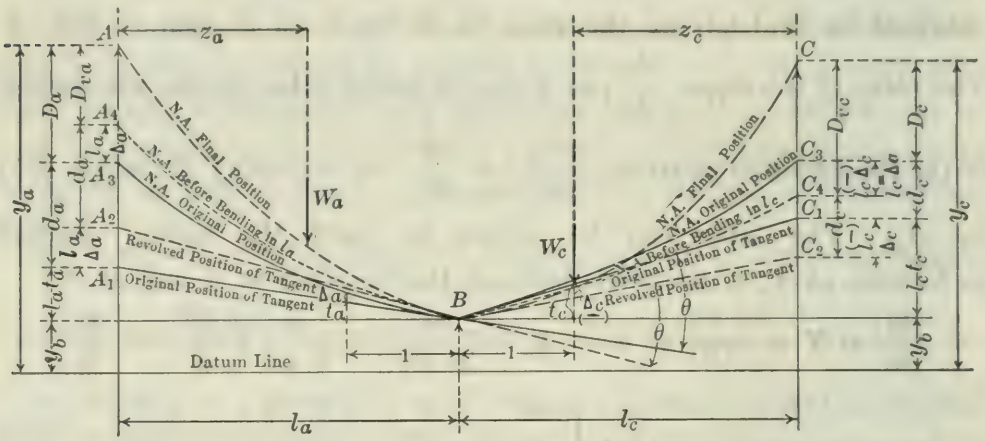

FIG. 3.

The deflection, $y_m - y$, at any point, X, cannot be obtained from the integrated value for D_v in Equation (12), because the origin for x is not at X. The bending moment, M, must first be written in terms of x_1 (say) with X as the origin, and the integration made by the General Equation, $D_v = \int \frac{Mx\,dx}{EI}$. If, however, the bending moment, M, is constant in the section considered, as, for instance, in the case of a simple beam loaded with two symmetrical concentrated loads, the deflection between the center and the load on either side can be obtained by Equation (12), because M has the same value regardless of the origin used.

SECTION 6.—FUNDAMENTAL GENERAL RELATION FOR CONTINUOUS BEAMS.

Consider two adjacent spans, l_a and l_c, of any continuous beam with a curved neutral axis (N. A. Fig. 3) resting only on the interme-

diate support, B, before the application of loads. Neglecting the effect of deflection due to shear, a fundamental equation based on the fact that the tangents to the neutral axes at B, for the two adjacent spans, remain at a constant fixed angle with each other, may be derived as follows:

Let t_a = the tangent of the angle between the original position of the tangent at B for the span, l_a, and the horizontal.

Δ_a = the change in t_a due to the rotation of the tangent at B.

D_a = the vertical distance from the original position of the neutral axis to the support, A, + if upward.

d_a = the vertical distance at A between the original tangent and the original neutral axis.

D_{va} = the total vertical deflection of the neutral axis at A due to bending in the span, l_a, only, that is,

$$D_{va} = \sum_A^B \frac{Mnx}{EI}.$$

t_c, Δ_c, D_c, d_c, D_{vc} = the corresponding quantities for the span, l_c, as shown.

y_a, y_b, and y_c = the elevations of the supports, A, B, and C.

The vertical movement, D_a, at A, between the original and final positions of the neutral axis, is seen to be composed of two parts: First, the movement, $A_3\ A_4$, due to the rotation of the tangent at B; and second, the movement or deflection, $A_4\ A$, due to the bending in the span, l_a, with respect to the tangent at B. This rotation of the tangent at B would certainly occur in the general case. The bending deflection for the span, l_a, only, is not equal to the total movement, D_a, except for the special case of symmetry about B such that no movement of the tangents at that point would take place.

From a consideration of Fig. 3, it is seen that the vertical movement at A between the original and final positions of the tangent is $A_1\ A_2 = l_a\ \Delta_a$; and the corresponding movement of the neutral axis vertically at A is $A_3\ A_4$, and also $= l_a\ \Delta_a$, because d_a remains practically constant. The bending deflection, $D_{va} = A_4\ A$, in the span, l_a, is the other portion of the movement, $A_3\ A$, of the neutral axis to bring it to the support at A. This deflection, by Equation (8), is:

$$D_{va} = \sum_A^B \frac{Mnx}{EI}.$$

If the $+ M$ terms are greater than the $- M$ terms, D_{va} is $+$ or upward, as shown. Note that the origin is at A.

As θ is constant:

$$\Delta_a = -\Delta_c \quad \ldots\ldots (19)$$

and, from Fig. 3,

$$D_{va} = D_a - l_a \Delta_a \quad \ldots\ldots (20)$$

Also,
$$D_{vc} = D_c - l_c \Delta_c = D_c + l_c \Delta_a \quad \ldots\ldots (21)$$

Solve Equations (20) and (21) for Δ_a, and equate:

$$\Delta_a = \frac{D_a - D_{va}}{l_a} = \frac{D_{vc} - D_c}{l_c}$$

whence,
$$\frac{D_a}{l_a} + \frac{D_c}{l_c} = \frac{D_{va}}{l_a} + \frac{D_{vc}}{l_c} \quad \ldots\ldots (22)$$

Substituting the values of D_{va} and D_{vc}:

$$\frac{D_a}{l_a} + \frac{D_c}{l_c} = \frac{\sum_A^B \frac{Mnx}{EI}}{l_a} + \frac{\sum_C^B \frac{Mnx}{EI}}{l_c} \quad \ldots\ldots (23)$$

Equation (23) is the fundamental and perfectly general equation by which the theorem of three moments can be developed by determining values for the summations indicated in the second member. As a check on Equations (22) and (23), consider the usual case of a horizontal beam resting on three supports, A B C. Assume a single load, W, on one span only; the final position of the neutral axis and the tangent at B would be as shown in Fig. 4. For this case, $D_a = 0$, $D_c = 0$, and Equation (22) gives:

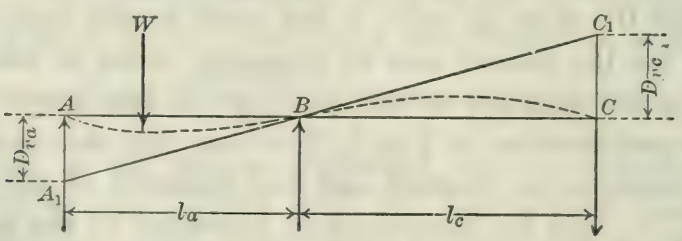

FIG. 4.

$$0 = \frac{D_{va}}{l_a} + \frac{D_{vc}}{l_c}; \text{ whence, } \frac{D_{va}}{l_a} = -\frac{D_{vc}}{l_c}.$$

This agrees with the result of considering the similar triangles, AA_1 B and BCC_1, as D_{vc} would be negative.

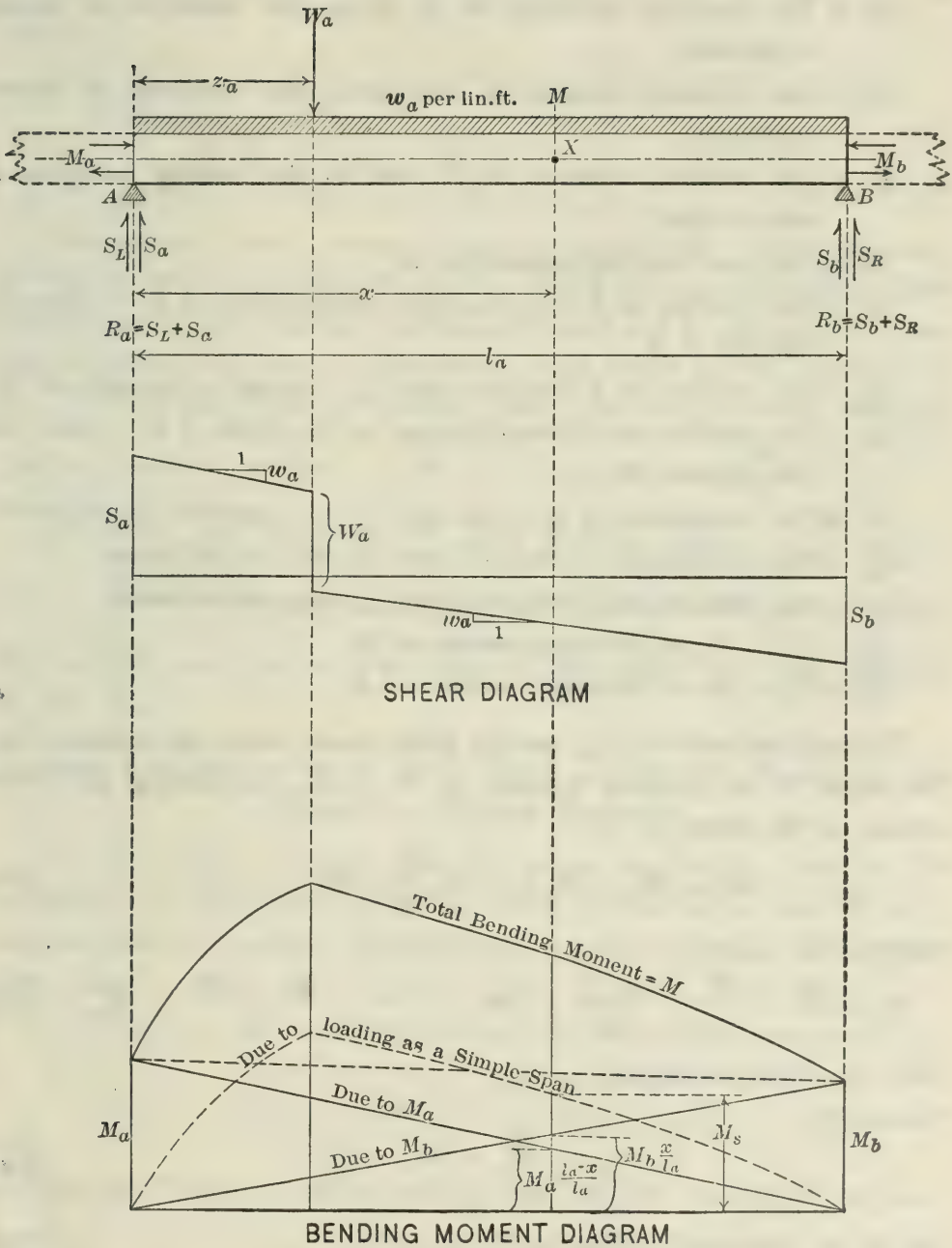
W_a
z_a
w_a per lin.ft.
M
X
M_a
M_b
A
B
S_L
S_a
S_b
S_R
x
$R_a = S_L + S_a$
$R_b = S_b + S_R$
l_a
1
w_a
S_a
W_a
S_b
SHEAR DIAGRAM
Total Bending Moment = M
Due to loading as a Simple Span
Due to M_a
Due to M_b
M_a
M_b
M_s
$M_b \frac{x}{l_a}$
$M_a \frac{l_a - x}{l_a}$
BENDING MOMENT DIAGRAM

FIG. 5.

SECTION 7.—GENERAL VALUES OF BENDING MOMENTS, SHEARS, AND REACTIONS, FOR ANY SPAN OF A CONTINUOUS BEAM.

Consider any span, AB, of a continuous beam, Fig. 5, and let:

M_a = the bending moment at A (from the theorem of three moments).

M_b = the bending moment at B (from the theorem of three moments).

M_s = the bending moment at X, due to the loading on l_a as a simple span.

M = the total bending moment at X.

S_L = the reaction at A from the span to the left of A = — shear to the left of A.

S_a = the reaction at A from the span, l_a = shear to the right of A.

S_R = the reaction at B from the span to the right of B = shear to the right of B.

S_b = the reaction at B from the span, l_a = — shear to the left of B

R_{sa} = the reaction at A from the span, l_a, as a simple beam.

R_{sb} = the reaction at B from the span, l_a, as a simple beam.

$R_a = S_L + S_a$ = the total reaction at A.

$R_b = S_b + S_R$ = the total reaction at B.

To find the value of S_a, for the given span, write the equation for the value of the bending moment at $B = M_b$, considering all forces acting on the span, l_a:

$$M_b = M_a + S_a l_a - w_a \frac{l^2_a}{2} - W_a (l_a - z_a) \dots\dots\dots (24)$$

Solving for S_a:

$$S_a = \frac{M_b - M_a}{l_a} + \frac{w_a l_a}{2} + W_a \frac{l_a - z_a}{l_a} = \frac{M_b - M_a}{l_a} + R_{sa} \dots (25)$$

Note that if $M_a = M_b$, $S_a = R_{sa}$.

Similarly:

$$S_b = \frac{M_a - M_b}{l_a} + R_{sb} \dots\dots\dots\dots\dots\dots\dots (26)$$

Or, S_b = (the total loading on l_a) — S_a .....(26a)

The values of M_a and M_b are first found from the theorem of three moments, and are generally negative. The sign must always be included in making substitutions in the foregoing formulas.

Having found the values of S_a and S_b for adjacent spans, the reactions are at once found by addition, noting that S_L = the S_b for the span on the left, and S_R = the S_a for that on the right.

The general value for M, at any point, X, can be stated in two ways:

First, $M = M_a + S_a\, x$ — (moment of loads on l_a to left of X about X).....(27)

Second, $M = \dfrac{M_a\,(l_a - x)}{l_a} + \dfrac{M_b\, x}{l_a} + M_s$.....(28)

This second form is seen to result at once from similar triangles in Fig. 4, and shows that M can be considered as composed of three distinct parts: (a) that due to M_a; (b) that due to M_b; and (c) that due to loads on l_a as a simple beam. The terms due to (a) and (b) can also be explained by noting that these end bending moments produce vertical couples; $+ M_a$ produces a force $+ \dfrac{M_a}{l_a}$ at B, and $- \dfrac{M_a}{l_a}$ at A, and similarly, M_b gives a force couple $+ \dfrac{M_b}{l_a}$ at A, and $- \dfrac{M_b}{l_a}$ at B. These forces produced as vertical couples to balance M_a and M_b also explain the form of Equations (25) and (26) for S_a and S_b.

Section 8.—Derivation of the Usual Restricted Form of the Theorem of Three Moments.

For the usual practical case, the beam is straight, rests on the supports, may slope at an angle, B, with the horizontal, and E and I are assumed as constant throughout the entire length. For such a case, then:

$D_a = 0$, $D_c = 0$, $n = dx$ sec. B, and Equation (23) becomes:

$$0 = \frac{\int_0^{l_a} \frac{M\, x\, dx \text{ sec. } B}{EI}}{l_a} + \frac{\int_0^{l_c} \frac{M\, x\, dx \text{ sec. } B}{EI}}{l_c}$$

As sec. B, E, and I, are constant, they can be multiplied out, and

$$\int_0^{l_a} \frac{M\, x\, dx}{l_a} + \int_0^{l_c} \frac{M\, x\, dx}{l_c} = 0 \quad \text{.....(29)}$$

To integrate $\int_0^{l_a} M\, x\, dx$, substitute for M the general value from Equation (28),

$$\int_0^{l_a} M\,x\,dx = \frac{M_a}{l_a}\int_0^{l_a}(l_a - x)\,x\,dx + \frac{M_b}{l_a}\int_0^{l_a} x^2\,dx + \int_0^{l_a} M_s\,x\,dx. \quad (30)$$

The first term becomes, $\frac{M_a}{l_a}\left[\frac{l_a\,x^2}{2} - \frac{x^3}{3}\right] = \frac{M_a\,l_a^2}{6}$;

the second term becomes, $\frac{M_b}{l_a} \times \frac{l_a^3}{3} = \frac{2\,M_b\,l_a^2}{6}$.

The third term in M_s consists of three parts: First, the bending moment in the segment, z_a, due to the concentrated load, W_a, that is, $\frac{W_a\,(l_a - z_a)}{l_a}\,x$; second, the bending moment in the segment, $l_a - z_a$, due to the concentrated load, W_a, that is, $\frac{W_a\,z_a}{l_a}\,(l_a - x)$; and third, the bending moment at any point due to the uniform load; w_a, that is, $\frac{w_a\,x}{2}\,(l_a - x)$.

Using these values, the third term then becomes:

$$\int_0^{l_a} M_s\,x\,dx = \frac{W_a}{l_a}\left[\int_0^{z_a}(l_a - z_a)\,x^2\,dx + z_a\int_{z_a}^{l_a}(l_a - x)\,x\,dx\right] +$$

$$\frac{w_a}{2}\int_0^{l_a}(l_a - x)\,x^2\,dx = \frac{W_a}{l_a}\left[(l_a - z_a)\,\frac{z_a^3}{3}\right.$$

$$\left. + z_a\left(l_a\,\frac{x^2}{2} - \frac{x^3}{3}\right)_{z_a}^{l_a}\right] + \frac{w_a}{2}\left[l_a\,\frac{x^3}{3} - \frac{x^4}{4}\right]_0^{l_a} = \frac{W_a}{l_a}\left[\frac{l_a\,z_a^3}{3}\right.$$

$$\left. - \frac{z_a^4}{3} + \frac{z_a\,l_a^3}{2} - \frac{z_a\,l_a^3}{3} - \frac{l_a\,z_a^3}{2} + \frac{z_a^4}{3}\right] + \frac{w_a\,l_a^4}{24}$$

$$\int_0^{l_a} M_s\,x\,dx = \frac{W_a\,z_a}{6}\,(l_a^2 - z_a^2) + \frac{w_a\,l_a^4}{24} \quad \ldots\ldots\ldots (31)$$

Substituting the foregoing values in Equation (30):

$$\int_0^{l_a} M\,x\,dx = \frac{M_a\,l_a^2 + 2\,M_b\,l_b^2 + W_a\,z_a\,(l_a^2 - z_a^2) + \frac{w_a\,l_a^4}{4}}{6} \quad \ldots (32)$$

By analogy, for the span, l_c;

$$\int_0^{l_c} M\,x\,dx = \frac{M_c\,l_c^2 + 2\,M_b\,l_c^2 + W_c\,z_c\,(l_c^2 - z_c^2) + \frac{w_c\,l_c^4}{4}}{6} \quad \ldots (34)$$

Substituting these values in Equation (29), eliminating the factor, 6, and transposing:

$$M_a\,l_a + 2\,M_b(l_a + l_c) + M_c\,l_c = -\,\Sigma\,W_a\,\frac{z_a}{l_a}\,(l_a^2 - z_a^2)$$

$$-\,\Sigma\,W_c\,\frac{z_c}{l_c}\,(l_c^2 - z_c^2) - \frac{w_a\,l_a^3}{4} - \frac{w_c\,l_c^3}{4} \quad \ldots\ldots (35)$$

The terms in W_a and W_c have the summation sign introduced in order to account for all such concentrated loads on the spans. Equation (35) is the usual restricted form of the theorem of three moments, and is used in general for all cases of continuous beams, even for swing-bridge trusses, where the neutral axis is not straight and the moment of inertia is not constant.

Section 9.—Effect of Settlement of Supports.

The foregoing form of the theorem assumes that the beam just rests on rigid supports. If the supports should settle, the values, D_a and D_c, would no longer be zero, but would be equal to the amount of such relative settlement. Assuming the beam to be straight, but having a slope, B, with the horizontal, Equation (23) becomes (as EI and sec. B are constant, n being equal to dx sec. B):

$$\frac{EI}{\text{sec. } B}\left[\frac{D_a}{l_a}+\frac{D_c}{l_c}\right]=\int_0^{l_a}\frac{M\,x\,dx}{l_a}+\int_0^{l_c}\frac{M\,x\,dx}{l_c} \quad \ldots.(36)$$

The second member of this equation is the same as that in Equation (29), which has been integrated in Equation (35). Thus, the only difference between Equation (35) and this case, is that the term, $\frac{6\ EI}{\text{sec. } B}\left[\frac{D_a}{l_a}+\frac{D_c}{l_c}\right]$, must be added, the factor, 6, having been multiplied out in getting Equation (35). The equation would then be:

$$M_a\,l_a+2\,M_b\,(l_a+l_c)+M_c\,l_c=-W_a\,\frac{z_a}{l_a}\,(l_a^2-z_a^2)$$

$$-W_c\,\frac{z_c}{l_c}\,(l_c^2-z_c^2)-\frac{w_a\,l_a^3}{4}-\frac{w_c\,l_c^3}{4}+\frac{6\ EI}{\text{sec. } B}\left[\frac{D_a}{l_a}+\frac{D_c}{l_c}\right]. \quad .(37)$$

This last term would be large, as it contains the multiplier, $E\ I$, and would be additive in effect if D_a and D_c were negative, that is, when the end supports settled downward with respect to the intermediate support.

All computations for bending moments, M, are made in foot-pounds or foot-kips, therefore the last term should be put in form for such computation by dividing by 12^2, as E and I are in inches, and D_a and D_c are in feet. Therefore, the last term of Equation (37) becomes:

$$\frac{EI}{24\text{ sec. } B}\left[\frac{D_a}{l_a}+\frac{D_c}{l_c}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(37a)$$

For the usual case of a horizontal beam, sec. $B = 1$.

Section 10.—Derivation of the Complete General Form of the Theorem of Three Moments.

The most general form of the theorem of three moments, applicable to any curved continuous beam with a variable moment of inertia and not originally resting on all supports, can be derived directly from the fundamental Equation (23). It is only necessary to obtain a general working form for the summations appearing in the second member of that equation, as follows:

Using the general value for M, as derived for Equation (18), there results:

$$\sum_A^B \frac{Mn\,x}{EI} = \sum_A^B \left[\frac{M_a}{l_a}(l_a - x) + \frac{M_b\,x}{l_a} + M_s\right]\frac{n\,x}{EI} \quad \ldots (38)$$

Let $n = dx \sec.\ \beta$, and assume E to be constant, then:

$$\sum_A^B \frac{Mn\,x}{EI} = \frac{1}{l_a E}\left[M_a \sum_A^B (l_a - x)\,x\,dx \frac{\sec.\ \beta}{I} + M_b \sum_A^B x^2\,dx \frac{\sec.\ \beta}{I} + l_a \sum_A^B M_s\,x\,dx \frac{\sec.\ \beta}{I}\right] \ldots\ldots (39)$$

By analogy, for span, l_c:

$$\sum_C^B \frac{Mn\,x}{EI} = \frac{1}{l_c E}\left[M_c \sum_C^B (l_c - x)\,x\,dx \frac{\sec.\ \beta}{I} + M_b \sum_C^B x^2\,dx \frac{\sec.\ \beta}{I} + l_c \sum_C^B M_s\,x\,dx \frac{\sec.\ \beta}{I}\right] \ldots\ldots (40)$$

By substituting the foregoing values in Equation (23), the following General Equation results:

$$E\left(\frac{D_a}{l_a} + \frac{D_c}{l_c}\right) = \frac{M_a}{l^2_a}\sum_A^B (l_a - x)\,x\,dx \frac{\sec.\ \beta}{I} + M_b\left(\frac{\sum_A^B x^2\,dx \frac{\sec.\ \beta}{I}}{l^2_a} + \frac{\sum_C^B x^2\,dx \frac{\sec.\ \beta}{I}}{l^2_c}\right) + \frac{M_c}{l^2_c}\sum_C^B (l_c - x)\left(x\,dx \frac{\sec.\ \beta}{I}\right) + \frac{\sum_A^B M_s\,x\,dx \frac{\sec.\ \beta}{I}}{l_a} + \frac{\sum_C^B M_s\,x\,dx \frac{\sec.\ \beta}{I}}{l_c} \quad \ldots (41)$$

This equation is the most general form of the theorem of three moments. Whenever the variables, sec. β and I, can be expressed as continuous functions of x, the summation sign can be replaced by the integration sign, and the mathematical integration can be made. Practically, it is generally found impossible to express these variables as continuous functions of x, and then make the integrations

Usually, I is constant for short sections of the beam, in which case it is best to make successive summations for such sections, using the general form of the integration, with I constant, and making the limits correctly account for the variation in I for different sections. This method will now be illustrated by a practical example.

Section 11.—Application to Plate-Girder Draw Span with Variable Moment of Inertia.

As an example of the practical method of using the general Equation (41), and also in order to determine the effect of the variable moment of inertia, consider the following case of a plate-girder span continuous over three supports. This is the same girder as used by Johnson, Bryan, and Turneaure* to illustrate the use of a similar formula obtained by applying the principle of least work. It is used here in order to find how closely the results of the different methods agree.

Data for Girder.—$l = l_a = l_c = 60$ ft., that is, the total length $= 120$ ft. The girder is symmetrical about its center line.

The relative values of the moment of inertia, I, at the following distances from the outside ends, are:

0 to 12 ft., $I = 1$
12 to 36 ft., $I = 1.38$
36 to 48 ft., $I = 1.15$
48 to 60 ft., $I = 1.92$

The girder rests on all the supports, and has a straight horizontal neutral axis, therefore, $D_a = 0$, $D_c = 0$, sec. $\beta = 1$.

The end bending moments are zero, therefore, $M_a = 0$, $M_c = 0$.

Noting the fact of the constant, I, for the given sections, let x_1 and x_2 be the distances from the end of the girder to the beginning and the end, respectively, of a given section having a constant, I, and let $\frac{1}{I} = i$ for such section. Then, by substituting in the General Equation (41), there results, after multiplying through by l^2:

$$0 = M_b \left[\sum_A^B \int_{x_1}^{x_2} i\, x^2\, dx + \sum_C^B \int_{x_1}^{x_2} i\, x^2\, dx \right] + l \left(\sum_A^B i\, M_s x\, dx + \sum_C^B i\, M_s x\, dx \right) \ldots\ldots\ldots (42)$$

* "Modern Framed Structures," Vol. II, p. 39.

For the case here considered, with a fully loaded beam carrying a uniform load, w lb. per ft., $M_s = \frac{w}{2} x (l - x)$. Therefore,

$$\sum i M_s x \, dx = \frac{w}{2} \sum \int_{x_1}^{x_2} i x^2 (l - x) \, dx$$

$$= \frac{w}{2} \sum \left[i x^3 \left(\frac{l}{3} - \frac{x}{4} \right) \right]_{x_1}^{x_2}$$

Then Equation (42) will give the following value for M_b:

$$M_b = - \frac{3 wl}{2} \frac{\sum_0^l \left[i x^3 \left(\frac{l}{3} - \frac{x}{4} \right) \right]_{x_1}^{x_2}}{\sum_0^l \left[i x^3 \right]_{x_1}^{x_2}} \quad \ldots\ldots\ldots (43)$$

The computation can best be made in the form shown by Table 1.

TABLE 1.—Computation for M_b.

6-ft. Sections.	(1) x_1	(2) x_2	(3) I	(4) i	(5) x^3	(6) $\left[i x^3 \right]_{x_1}^{x_2}$		(7) $\frac{l}{3} - \frac{x}{4}$	(8) $\left[i x^3 \left(\frac{l}{3} - \frac{x}{4} \right) \right]_{x_1}^{x_2}$	
						Columns 4 × 5			Columns 6 × 7	
1-2	0	12	1.0	1.0	x_2 1 730	1 730	.....	17	29 400	
					x_1 0	— 0	1 730	20	0	29 400
3-4-5-6	12	36	1.38	0.725	x_2 46 700	33 700	.....	11	371 000	
					x_1 1 730	— 1 200	32 500	17	— 21 000	350 000
7-8	36	48	1.15	0.87	x_2 110 600	96 000	.....	8	768 000	
					x_1 46 700	—40 400	55 600	11	— 445 000	323 000
9-10	48	60	1.92	0.52	x_2 216 000	112 400	.....	5	562 000	
					x_1 110 600	—57 400	55 000	8	— 460 000	102 000
						Σ = 144 800			Σ = 804 400	

Then, $$M_b = - \frac{3}{2} wl \times \frac{804\,400}{144\,800} = - 8.34 \, wl.$$

For the given case, $l = 60$ ft.:

Therefore, $M_b = -8.34 \times 60\ w = -500\ w$.

When I is assumed as constant, as in the usual form of the theorem, let $l_a = l_c = l$, and $w_a = w_c = w$, in Equation (35),

$$2\ M_b\ (l + l) = -2\left(\frac{wl^3}{4}\right), \text{ whence,}$$

$$M_b = -\frac{wl^2}{8} = \frac{-w \times 60^2}{8} = -450\ w.$$

The exact method, therefore, gives a value for M_b about 11% higher than the approximate method, using the constant, I. The method of least work* gave $M_b = -492\ w$, about 2% less than the value here found by the exact method.

To Derive the Maximum Limit for Error.—The error introduced by using the general form of the theorem of three moments in such cases of girders with variable I, carrying the uniform load, and symmetrical over three supports, can be shown to be about 15% for the limiting case where I is assumed to vary directly with M, that is, for a theoretically perfect design, which, of course, is a practically impossible case.

The reaction, R_a, can be found by locating the point of contraflexure where the bending moment changes sign, and noting that the end shears for the equivalent simple beam, between points of contraflexure or zero bending moment, can be found exactly as in simple beam analysis. For the foregoing case of two equal spans loaded with equal uniform loads, the tangent at the intermediate support, B, remains fixed and horizontal. Therefore, if x_0 = distance from A to the point of contraflexure:

$$[D_{va}]l_0 = \int_0^l \frac{M\,x\,dx}{EI} = \int_0^{x_0} \frac{M\,x\,dx}{EI} - \int_{x_0}^l \frac{M\,x\,dx}{EI} = 0.$$

Now let $\frac{M}{I} = K$, substitute, and cancel the constants, $\left(\frac{K}{E}\right)$,

$$\int_0^{x_0} x\,dx = \int_{x_0}^l x\,dx$$

$$\frac{x_0^2}{2} = \frac{l^2}{2} - \frac{x_0^2}{2}. \quad \text{Therefore, } x_0 = \frac{l}{\sqrt{2}} = 0.707\ l.$$

(Note that $x_0 = 0.75\ l$, when I is assumed as constant.)

Then, $R_a = \frac{wx_0}{2} = 0.354\ wl$; and, $M_b = R_a\ l - \frac{wl^2}{2} = 0.354\ wl^2 - 0.5\ wl^2 = -0.146\ wl^2$.

* As applied in "Modern Framed Structures," Vol. II, p. 40.

As shown before, the usual form of theorem, with I assumed as constant, gives for this case:

$$M_b = -\frac{wl^2}{8} = -0.125\ wl^2.$$

Therefore, the approximate method gives results which are too low by a maximum limit of 15% of the exact value, or 16.8% of the approximate value.

The general conclusion that the usual approximate formula will give M_b from 10 to 15% too low, when applied to girder spans with widely varying moments of inertia, seems to be quite justifiable, the effect of shear deflection being neglected.

Two cases of swing-bridge girders with variable I are solved by C. W. Hudson, M. Am. Soc. C. E., by the formulas derived from the method of deflections.* The first is a light single-track girder with two equal spans of 68 ft.; and the second is a heavy double-track girder with two equal spans of 78 ft. In the first case the center bending moment, M_b, is 15% greater, and in the second case 11% greater, by the exact method than by the approximate method. Mr. Hudson,† also computes the effect of shear deflection on the center bending moment, and finds, in the first case, a decrease of 2.6%, and in the second a decrease of 5%, as compared with the value obtained by neglecting shear distortion.

It is evident that the usual approximate assumption, neglecting the variable I, and shear deflection, will introduce errors which tend to compensate each other. Especially is this true in the case of framed trusses, where the effect of shear distortion is relatively much greater. It has been found elsewhere that the usual method of designing swing-bridge trusses by the ordinary restricted formula, assuming I as constant and neglecting shear deflection, is justifiable, because the method of deflections, when applied to the resulting design, gives reactions which agree very closely with those found by the usual approximate method using the restricted form of the theorem of three moments. For plate-girder draw spans, however, the ordinary design formulas have been shown to introduce errors as large as from 10 to 12% on the side of danger, when the effects of variable moments of inertia are neglected.

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* "Deflections and Statically Indeterminate Stresses," pp. 49–50.

† *Loc. cit.*, pp. 54-55.

DISCUSSION

Mr. Ketchum.

MILO S. KETCHUM, M. AM. SOC. C. E. (by letter).—The writer wishes to call attention to the "method of area moments" for the solution of statically indeterminate structures. This method, due to Mohr, is much easier to derive and shorter to apply to the calculation of constrained beams than either the "theorem of three moments" or the "method of least work." The author is in error in referring to the method of three moments as being an exact one. It is subject to the same limitations as the methods of least work and area moments, and all methods, if correctly applied, will give identical results.

Before applying the method of area moments to the problem of the plate-girder draw span with variable moment of inertia, it will be necessary to discuss briefly the method. The statement of the method of area moments is:

"If A and B are two points in a beam, the deflection of B with respect to a tangent at A, is equal to the moment about B of the area of the portion of the equilibrium polygon between A and B, multiplied by $\frac{H}{EI}$, where E is the modulus of elasticity of the material, I is the moment of inertia of the section of the beam, and H is the pole distance of the force polygon that was used in drawing the equilibrium polygon."

The proof of this statement, together with applications, has been given by the writer.* If the equilibrium polygon is drawn with a force polygon having a unit pole, the equilibrium polygon becomes the true bending moment polygon, and the statement of the principle is simplified accordingly.

The application of the method of area moments to a simple beam with a constant moment of inertia, loaded with a concentrated load and with a uniform load, respectively, is shown in the following problems:†

Simple Beam.—Concentrated Load at Center of Beam.—The simple beam in (*a*), Fig. 6, is loaded with a load, P, at the center. The bending moment diagram is shown in (*b*), and the beam is loaded with the bending moment diagram in (*c*), Fig. 6.

To find the equation of the elastic curve, take moments of the forces to the left of a point at a distance, x, from the left support, and the deflection of the beam at any point in the beam, (*a*), Fig. 6, will be equal to the bending moment at the corresponding point in the beam, (*b*), Fig. 6, divided by EI, and

$$-EIy = \frac{PL^2x}{16} - \frac{Px^3}{12} \quad \ldots\ldots(1)$$

and

$$48\,EIy = P\,(4\,x^3 - 3\,L^2x) \quad \ldots\ldots(2)$$

* "Steel Mill Buildings," Chapter XIVa; and "Mine Structures," Chapter IV.

† Taken from "Steel Mill Buildings."

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The maximum deflection will occur when $x = \frac{1}{2} L$, in Equation (2), or it may be found by taking moments of forces to the left of $x = \frac{1}{2} L$, to be

$$\Delta = \frac{PL^3}{48\ EI} \qquad (3)$$

Beam Uniformly Loaded.—The simple beam in (*a*), Fig. 7, is loaded with a uniform load of w per linear foot. The bending moment parabola is shown in (*b*), and the beam is loaded with the bending moment parabola in (*c*), Fig. 7. To find the equation of the elastic curve, take moments of forces to the left of a point at a distance, x, from the left support.

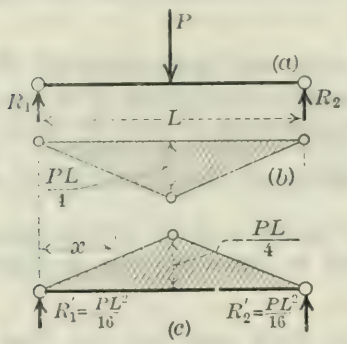

FIG. 6.

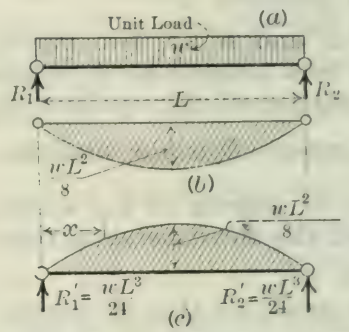

FIG. 7.

The equation of the bending moment parabola with the origin of co-ordinates at the left support is $y = \frac{1}{2} wLx - \frac{1}{2} wx^2$; the area of a segment of the parabola is

$$A = \frac{1}{4} wLx^2 - \frac{1}{6} wx^3 \qquad (4)$$

and the center of gravity measured back from x is

$$-X_1 = \frac{x\ (2\ L - x)}{6\ L - 4x} \qquad (5)$$

Taking moments of forces to the left of a point, x, and reducing, we have

$$24\ EIy = w\ (-x^4 + 2\ Lx^3 - L^3x) \qquad (6)$$

The deflection is a maximum when $x = \frac{1}{2} L$, and may be found directly by taking moments, or may be found from Equation (6), and is

$$\Delta = \frac{5\ wL^4}{384\ EI} \qquad (7)$$

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For a beam with a variable moment of inertia, the transformed beams, (*b*), Figs. 6 or 7, should be loaded with the bending moment divided by the moment of inertia, I, at each point, or with the bending moment divided by EI at each point, if both the modulus of elasticity and the moment of inertia are variable.

The reaction of the center pier of a draw span of two equal spans with a constant moment of inertia and modulus of elasticity, may be calculated as follows:

Draw Span with Constant Section.—If the total span be $2\ l = L$, in Fig. 6, the deflection of the beam with span, L, loaded with a uniform load, w pounds per linear foot, is

$$\Delta = \frac{5\ wL^4}{384\ EI} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Now, if the center reaction, R_b, be considered as a load acting upward, it will be equal to a load, P, acting at the center of the span, which produces a deflection equal to the deflection of the simple beam, in Fig. 7, loaded with a load, w. The deflection of the beam loaded with a load, P, is

$$\Delta = \frac{PL^3}{48\ EI} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Equating the values of Δ in Equations (7) and (8) gives the middle reaction

$$R_b = P = \frac{5}{8}\ wL \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

With the foregoing preliminary statement, we may now take up the solution of the problem of the plate-girder draw span with variable moment of inertia, which was solved by the author in Section 11.

Plate-Girder Draw Span.—The span $l_a = l_c = 60$ ft.; the total length is $l = 120$ ft. The girder is symmetrical about its center line. The relative values of the moments of inertia, I, at the following distances from the outside ends are:

0 to 12 ft., $I = 1$
12 to 36 ft., $I = 1.38$
36 to 48 ft., $I = 1.15$
48 to 60 ft., $I = 1.92$

Solution.—Take a simple beam with a span of $l = 120$ ft., having the same cross-section as the draw span, and loaded with a uniform load of w pounds per linear foot, as in (*a*), Fig. 8. The bending moment diagram will be a parabola with a maximum ordinate at the center, $M = \frac{1}{8}\ wl^2 = 1\,800\ w$. To calculate the deflection in the beam due to the load, w, load the transformed beam in (*b*), Fig. 8, with $\frac{M}{I}$.

Mr. Ketchum.

The areas of the segments of the parabola in (*b*), Fig. 8, were calculated by Equation (4), and are as follows:

$$A_1 = 4\,032\ w \text{ (square-foot-pounds.)}$$
$$A_2 = 19\,618\ w \quad " \quad " \quad "$$
$$A_3 = 17\,030\ w \quad " \quad " \quad "$$
$$A_4 = 11\,100\ w \quad " \quad " \quad "$$

The centers of gravity of the segments of the parabola were calculated by Equation (5) as follows:

The center of gravity of A_1 about *a* is

$$x_1 = \frac{x\,(240 - x)}{720 - 4x} = \frac{12\,(240 - 12)}{720 - 48} = 4.07 \text{ ft.},$$

and the distance of the center of gravity of A_1 from the center of the beam, *B*, is 48 ft. + 4.07 ft. = 52.07 ft.

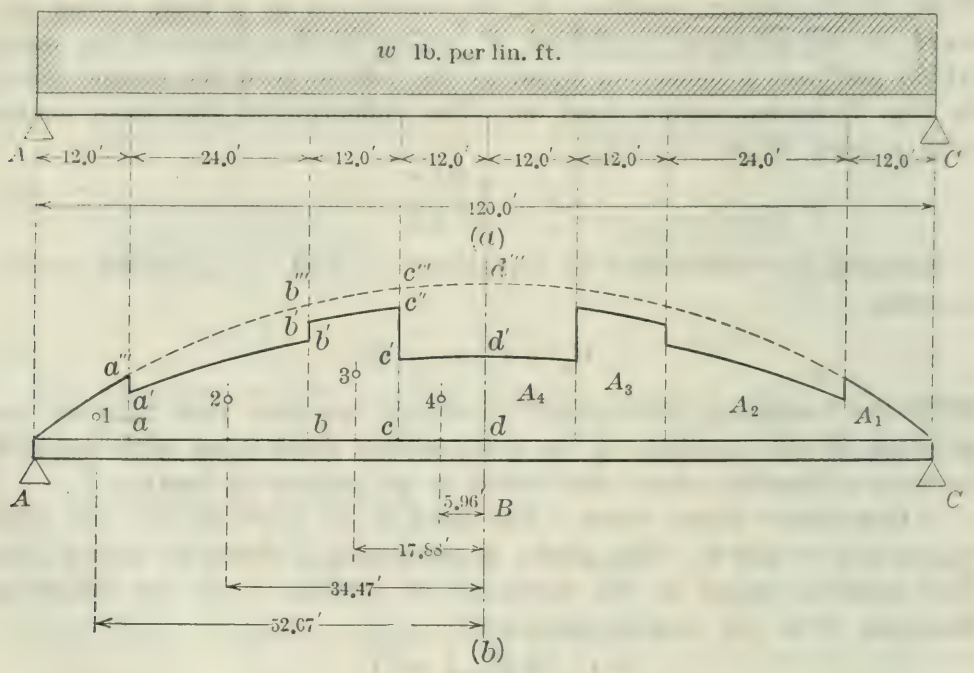

FIG. 8.

The center of gravity of the part of the dotted parabola about the point, *b*, is

$$\bar{x} = \frac{x\,(240 - x)}{720 - 4x} = \frac{36\,(240 - 36)}{720 - 144} = 12.75 \text{ ft.}$$

The center of gravity of the dotted A_2 about *b* will be

$$x_2 = \frac{(\text{Area } A_1 + \text{Area } A_2)\,12.75 \text{ ft.} - \text{Area } A_1 \times 28.07 \text{ ft.}}{\text{Area } A_2}$$

$$= \frac{31\,104\ w \times 12.75 \text{ ft.} - 4\,032\ w \times 28.07 \text{ ft.}}{27\,072\ w}$$

$$= 10.47 \text{ ft.}$$

and the distance of the center of gravity of A_1 from the center of the beam, B, is 48 ft. + 4.07 ft. = 52.07 ft. Mr. Ketchum.

The centers of gravity of A_3 and A_4 are calculated in the same manner.

The bending moment at the center of the beam, (b), Fig. 8, will be E times the deflection at the center of the beam, (a), Fig. 8.

$$M = E\ \Delta = R_1 \times 60 \text{ ft.} - A_1 \times 52.07 \text{ ft.} - A_2 \times 34.47 \text{ ft.} - A_3 \times 17.88 \text{ ft.} - A_4 \times 5.96 \text{ ft.}$$
$$= 51\,780\,w \times 60 \text{ ft.} - 4\,032\,w \times 52.07 \text{ ft.} - 19\,618\,w \times 34.47 \text{ ft.} - 17\,030\,w \times 17.88 \text{ ft.} - 11\,100\,w \times 5.96 \text{ ft.}$$
$$= 1\,850\,340\,w \text{ (cubic-foot-pounds)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Now load the same beam with a load of 1 lb., as in (a), Fig. 9. The bending moment diagram is a triangle with a center height of $M = \frac{Pl}{4} = 30$ ft-lb. The beam is loaded with the bending moment divided by the moment of inertia at each point.

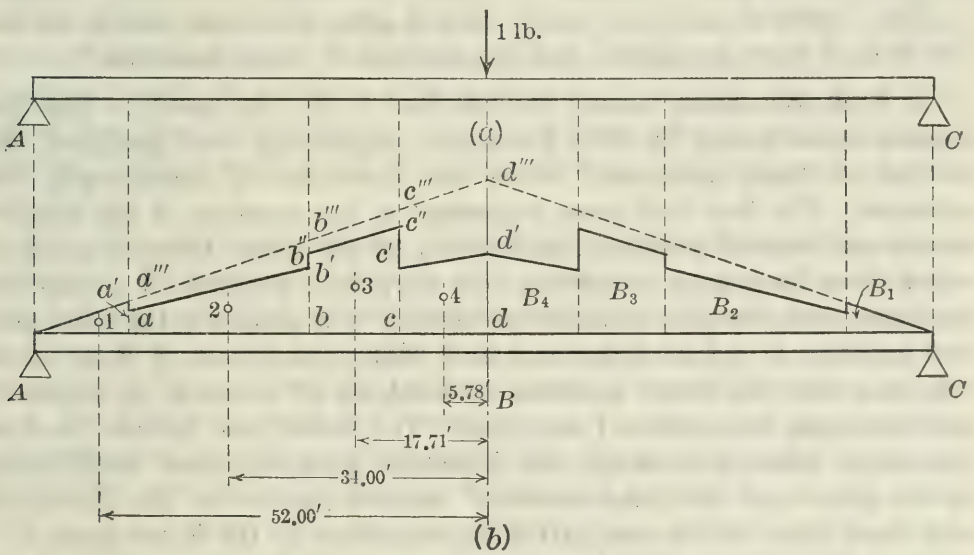

FIG. 9.

The areas of the different sections of the moment diagram are:

$$B_1 = 36 \text{ (square-foot-pounds.)}$$
$$B_2 = 208.7 \text{ " " "}$$
$$B_3 = 219.1 \text{ " " "}$$
$$B_4 = 168.7 \text{ " " "}$$

The centers of gravity are as shown in (b), Fig. 9. Then the bending moment at the center of the beam (b), Fig. 9, will be E times the deflection of the beam (a), Fig. 9, due to a center load of 1 lb.

Mr. Ketchum.

$$M = E\,J' = R_1 \times 60 \text{ ft.} - B_1 \times 52 \text{ ft.} - B_2 \times 34.00 \text{ ft.} - B_3 \times 17.71 \text{ ft.} - B_4 \times 5.78 \text{ ft.}$$
$$= 632.5 \times 60 \text{ ft.} - 36 \times 52 \text{ ft.} - 208.7 \times 34.00 \text{ ft.} - 219.1 \times 17.71 \text{ ft.} - 168.7 \times 5.78 \text{ ft.}$$
$$= 24\,137 \text{ (cubic-foot-pounds)} \quad \ldots\ldots (11)$$

Now the center reaction of the draw span will be equal to Equation (10) divided by Equation (11), and

$$R_2 = 1\,850\,340\ w \div 24\,127$$
$$= 76.67\ w.$$

The left reaction is

$$R_1 = \tfrac{1}{2}\ (120\ w - 76.67\ w)$$
$$= 21.665\ w.$$

The bending moment at the center support of the draw span is

$$M_b = 21.655\ w \times 60 \text{ ft.} - 60\ w \times 30 \text{ ft.}$$
$$= 500.1\ w \text{ (foot-pounds).}$$

This value checks the value calculated by the theorem of three moments.

The "method of least work" should give the same result as the "method of three moments" and the method of "area moments."

Mr. Williams.

J. P. J. WILLIAMS, ASSOC. M. AM. SOC. C. E. (by letter).—The discussion contributed by Mr. Ketchum, explaining and applying the method of "area moments," raises two questions of considerable importance. The first and more important is the question of the relative merits and ease of practical application of the three different methods which have been used in solving this particular problem of a continuous beam with variable moments of inertia. The second is the more general question of actual differences and relative exactness of these methods, and also the broad question of methods of solution of statically indeterminate structures of any kind. The writer also intends to show the direct relation between the deflection formulas used analytically in the paper and the "area moment" method applied by Mr. Ketchum, and check some of his own particular equations by the latter method.

In order to determine the relative amount of labor required to solve the problem of the plate-girder draw span with variable moments of inertia, the writer has counted the number of "operations" required to make the solution for the center bending moment, M_b, by each of the three methods. By "operation" is meant, either a setting and reading of the slide-rule, or an addition or subtraction, also making allowance for cases of relatively easy additions or multiplications. The "area moment" method applied by Mr. Ketchum required 105 such operations, the tabular computation made by Mr. Turneaure, using a formula in terms of bending moments, required 78 operations, and the three moment method, using the writer's Equation (43) and

Mr. Williams

Table 1, required only 41 operations. As the labor required to draw the diagrams for the "area moment" method is certainly as great as that of making the tables in the other methods, it is quite evident that, for this particular problem, the three moment method is far superior. It does not necessarily follow that, for other cases of different loading, or beams on more than three supports, the same method would still be best, but it is probable that it would, provided the mathematical integrations were handled rapidly.

The second question concerning the exactness of these three methods requires a clearer understanding of their essential differences. A careful reading of the first paragraph and the last three paragraphs of the paper will indicate that the term "exact method" used by the writer had reference only to the three moment method which included the effects of variable moments of inertia, settlement of supports, and slope of neutral curve. The principal object in view in Section 11 of the paper was the determination of the value of the error introduced in a particular case of a plate-girder draw span when the usual three moment formulas, neglecting effect of variable moments of inertia, were used; but, as Mr. Ketchum has raised the question of the relative exactness of the different "exact" methods, and has checked the writer's result so closely by using the "area moment" method, therefore indicating that the 2% discrepancy in Mr. Turneaure's result is probably inherent in that method, it is well to consider the different methods more fully.

Both Mr. Ketchum and Mr. Turneaure find the value of the center reaction by dividing the deflection of the girder assumed as simply supported at the ends (that is, with center reaction removed), and carrying the given load, by the deflection at the center caused by a unit load at that point. They differ only in the manner of computing these deflections, Mr. Ketchum using the "area moment" method and equations for area and positions of centroids, and Mr. Turneaure using a tabular summation method, assuming the differential length, dx, in the equations to have a finite value of 6 ft. The formulas are essentially identical, but the exactness of the latter method depends directly on the length assumed for dx, which, theoretically, should be very small. The 2% discrepancy between the result of this method and the results of the other two is probably due to this fact. The three moment method which the writer used is also essentially a deflection method, as it was based on the General Equation (41) derived by Equation (8) for vertical relative deflections of the ends with respect to a tangent to the neutral curve at the center. Mathematically, therefore, all three methods are based on the same theory of deflection due to bending, and, except for the practical inaccuracies in application, should give the same results. The vital differences in the working equations used are only differences in details of application.

Mr. Williams.

It should be emphasized, further, that all these methods give results which are inexact because of the neglect of shear deflection and of shear distortion of the assumed plane normal sections of the beam. Short, heavily-loaded girders with large concentrated loads are affected very materially by shear distortion, as has been shown in the writer's discussion* of "Faults in the Common Theory of Flexure"; and the common assumption of bending deflection only may be largely in error on the side of danger.

All statically indeterminate problems of any character can be solved by either of three general methods: (1) Method of Deflections or Deformations, (2) Method of Equal Work, and (3) Method of Least Work. All three methods give identical results when restricted to the same kinds of stress, but the first uses expressions for deflections or strains, the second equates internal work to external work, and the third applies the principle of minimum internal work.

For example, consider the very simple case of two elastic supports, a and b, subject only to the uniformly distributed direct stress caused by a load, W, carried in such a way that the total elastic strain of each support is the same.

Let W_a = portion of load, W, carried by a;
$W - W_a$ = portion of load, W, carried by b;

$$e_a = \frac{E_a A_a}{L_a} = \text{stiffness of support, } a, = \text{total force required}$$

to produce total strain of unity in a;

$$e_b = \frac{E_b A_b}{L_b} = \text{stiffness of support } b.$$

Then the total deflection or strain $= \dfrac{\text{stress}}{\text{stiffness}}$.

1.—Method of Deflections.—Equate deflection of a to deflection of b.

$$\frac{W_a}{e_a} = \frac{W - W_a}{e_b}; \text{ therefore } W_a = \frac{e_a}{e_a + e_b} W.$$

2.—Method of Equal Work.—Equate internal work to external work.

$$\frac{1}{2}\left[W_a \frac{W_a}{e_a} + W - W_a \frac{W - W_a}{e_b}\right] = \frac{1}{2} W \frac{W_a}{e_a}$$

$$\frac{W_a^2}{e_a} + \frac{(W - W_a)^2}{e_b} = \frac{W W_a}{e_a}$$

or $\dfrac{W_a}{e_a}(W - W_a) = \dfrac{(W - W_a)^2}{e_a}$; therefore $\dfrac{W_a}{e_a} = \dfrac{W - W_a}{e_b}$, as in 1

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 932-953.

3.—Method of Least Work.—Write the expression for internal work or resilience, R, and place the first derivative with respect to W_a equal to zero, to get condition for minimum. Mr. Williams.

$$R = \frac{1}{2}\left[\frac{W_a^2}{e_a} + \frac{(W - W_a)^2}{e_b}\right]$$

$$\frac{dR}{dW_a} = \frac{1}{2}\left[\frac{2\,W_a}{e_a} - \frac{2\,(W - W_a)}{e_b}\right] = 0; \text{ therefore}$$

$$\frac{W_a}{e_a} = \frac{W - W_a}{e_b}, \text{ as in 1.}$$

All three methods thus reduce to identical expressions, and it is possible to reduce even more complex problems, solved by the different methods, to the same working form. Usually, the method of deflection is the most direct and easiest of application. The vertical deflection of beams can be obtained in various ways: By the double integration of the fundamental equation, $M = EI\frac{d^2y}{dx^2}$, of the common theory of flexure; by the application of Equation (8) for $D_v = \sum_O^N \frac{Mn}{EI}x$; or by the semi-graphical method of area moments applied by Mr. Ketchum. They, also, are identical in result, if properly applied, and differ only in manner of procedure.

In order to show the relation between the last two methods, and also explain directly the "area moment" method in connection with that used in the paper, it is only necessary to note that $D_v = \sum_O^N \frac{Mn}{EI}\,x$ can be found by area moments. If values of $\frac{M}{EI}$ be plotted to scale as ordinates, and the lengths, n, be laid off on the X axis, then $\frac{Mn}{EI}$ will be an elementary area, and $\frac{Mn}{EI}x$ will be the moment of that area about the origin, O, the deflecting point. Then D_v = deflection of O with respect to a tangent to the neutral curve at $N = \sum_O^N \frac{Mn}{EI}x$ = total moment of area of diagram between limits O and N about O. Similarly, the change, A, in the angle between the tangents to the neutral curve at O and at N will be the area of the same diagram between limits O and N, because, by Equation (4), $A = \sum_O^N \frac{Mn}{EI} = \sum_O^N$ elementary areas = area between limits O and N. When E is constant, and M a continuous func-

Mr. Williams.

tion of x, n becomes dx for a horizontal beam, and if A_i = area of a diagram plotted with $\frac{M}{I}$ as ordinates and x_i = the distance of its centroid from the deflecting point as origin, then, for any given limits:

$$A = \frac{A_i}{E}, \text{ and } D_v = \frac{A_i x_i}{E}.$$

The last equation is the basis of the area moment method applied by Mr. Ketchum. When both E and I are constant, the ordinary bending moment diagram area can be used, because M is the ordinate and Mdx an elementary area.

Then
$$A = \frac{1}{EI}\int_O^N Mdx = \frac{A_m}{EI}$$

and
$$D_v = \frac{1}{EI}\int_O^N Mdx \,.\, x = \frac{A_m x_m}{EI}$$

in which A_m = area of bending moment diagram between limits O and N, and x_m = distance from centroid of A_m to origin O.

For example, the integration, $\int_0^{l_a} M\ dx\ x$, in Equation (30), can be more easily made by this method of area moments than by the analytic integration as given. Using areas of the bending moment diagram for M, Fig. 5, noting its component parts and that the origin is at A, the values of $A_m\ x_m$ will be:

For triangle due to M_a,

$$A_m x_m = \frac{M_a l_a}{2}\,\frac{l_a}{3} = \frac{M_a l^2{}_a}{6}$$

For triangle due to M_b,

$$A_m x_m = \frac{M_b l_a}{2}\,\frac{2\,l_a}{3} = \frac{2\,M_b\,l^2{}_a}{6}.$$

For triangle (not shown) due to bending moment caused by concentrated load, W_a, in simple beam, the area, A_m, would be:

$$A_m = \frac{W_a z_a}{l_a}(l_a - z_a)\,\frac{l_a}{2} = \frac{W_a z_a}{2}(l_a - z_a)$$

The distance from centroid to origin, A, is $x_m = \frac{l_a + z_a}{3}$.

Therefore, $A_m x_m = \frac{W_a z_a}{2}(l_a - z_a)\,\frac{l_a + z_a}{3} = \frac{W_a z_a}{6}(l^2{}_a - z^2{}_a)$.

as in Equation (31).

For the parabolic bending moment diagram (not shown) due to uniform load, w_a, on a simple beam, the maximum center ordinate being $\frac{w_a l_a{}^2}{8}$, the area $A_m = \frac{2}{3}\,\frac{w_a l_a{}^2}{8}\,l_a = \frac{w_a\,l_a{}^3}{12}$, and $x_m = \frac{l_a}{2}$.

Mr. Williams.

Therefore, $$A_m \, x_m = \frac{w_a \, l_a^3}{12} \frac{l_a}{2} = \frac{w_a \, l_a^4}{24},$$

as in Equation (31).

The "equivalent beam" loaded with the bending moment diagram is a convenient method for finding, not only the deflection, y, but the slope, $\frac{dy}{dx}$, at any point of a given beam. As it will be of interest to note how this method is developed from the general equations already applied, consider the case of a simple beam, with constant E and I, and loaded uniformly with w pounds per foot, as in Fig. 2. The end reaction of the "equivalent beam" loaded with the parabolic bending moment diagram, as in Fig. 7 (c), is,

$$\frac{1}{2}\left[\frac{2}{3}\frac{wl^2}{8}\, l\right] = \frac{wl^3}{24} = \text{area between end and center.}$$

This area between end and center, however, when divided by EI, is the angle between the tangent at the end and the tangent at the center, when E and I are constant. As the tangent at the center is horizontal, the slope of the end tangent with the horizontal, therefore, is $\frac{wl^3}{24\,EI}$, as already obtained analytically in Equation (16). Thus slope of tangent at A equals reaction of "equivalent beam" divided by EI.

From Fig. 2, the slope at any point, X is:

$$\frac{dy}{dx} = (\text{slope of tangent at } A) - A_x$$

$$= \frac{\text{reaction of "equivalent beam"} - \text{area between } A \text{ and } X}{EI}$$

$$= \frac{\text{shear of "equivalent beam"}}{EI}.$$

Hence the slope, $\frac{dy}{dx}$, at any point, X, is equal to the shear at that point of the "equivalent beam" divided by EI. Using Equation (4) of Mr. Ketchum's discussion for the area of the segment of the parabola in Fig. 7 (c):

$$\text{Area between } A \text{ and } X = \frac{wlx^2}{2} - \frac{wx^3}{6} = \frac{wx^2}{24}(6\,l - 4\,x)$$

Therefore $$\frac{dy}{dx} = \frac{wl^3}{24\,EI} - \frac{wx^2}{24\,EI}(6\,l - 4\,x)$$

$$= -\frac{w}{24\,EI}[6\,lx^2 - 4\,x^3 - l^3],$$

which checks Equation (17), previously found by integration methods.

Mr. Williams.

The deflection, y, at any point, X, Fig. 2, is seen to have the value:

$$y = (\text{slope of tangent at } A)\, x - \text{deflection of } X \text{ with respect to tangent at } A$$

$$= \frac{\text{reaction of ``equivalent beam''}}{EI}\, x - \frac{\text{moment of area, } A \text{ to } X, \text{ about } X}{EI}$$

$$= \frac{\text{bending moment of ``equivalent beam''}}{EI}.$$

Using $x_1 =$ distance from centroid of area, A to X, from the point, X,

$$x_1 = \frac{x\,(2\,l - x)}{6\,l - 4x},$$

from Equation (5), given by Mr. Ketchum, there results:

$$y = \frac{wl^3}{24\,EI}\,x - \frac{wx^2}{24\,EI}\,(6\,l - 4\,x)\,\frac{x\,(2\,l - x)}{6\,l - 4\,x}$$

$$= \frac{-w}{24\,EI}\,[2\,lx^3 - x^4 - l^3x],$$

which checks the writer's Equation (18), or Equation (6), with opposite sign, as obtained by Mr. Ketchum.

This "equivalent beam" method of finding slope and deflection is perfectly general, and can be applied to beams with variable moments of inertia by a simple modification of the bending moment diagram, as already explained. The application made by Mr. Ketchum to the plate-girder draw span with variable moments of inertia, considered in the paper in Section 11, is a good illustration of this method of using an "equivalent beam," but, as already stated, it is much more laborious for this particular case than the three moment method giving identical results. Mr. Ketchum could have decreased by twelve the number of operations required for his solution by having used, instead of bending moment of "equivalent beam," simply the sum of the area moments about the end, as such sum gives the deflection of the end above the horizontal tangent at the center, which is equal to the center deflection desired for this particular symmetrical case.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1259

CHARACTERISTICS OF CUP AND SCREW CURRENT METERS; PERFORMANCE OF THESE METERS IN TAIL-RACES AND LARGE MOUNTAIN STREAMS; STATISTICAL SYNTHESIS OF DISCHARGE CURVES.*

By B. F. Groat, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. G. Price, E. E. Haskell, Charles H. Miller, John C. Hoyt, C. W. Staniford, and B. F. Groat.

During the summer and autumn of 1911 the writer ran a number of efficiency tests on two of the 6 000-h.p. hydraulic turbine units recently installed in the power-house of the St. Lawrence River Power Company, at Massena, N. Y. On one of the units, two sets of tests were run, one prior, and one subsequent, to cutting off several feet of the draft-tubes. On the other unit, one set of tests was run after the draft-tubes had been shortened, as had been done in the case of the other turbine unit.

In all, about 40 000 instrumental readings, including gauge readings, were made. Of these, 7 000 were complete velocity observations by current meters, and 4 000 additional readings on a Pitot tube furnished about 100 complete velocity observations for comparison with simultaneous readings by the current meters.

Owing to the nature of the tail-races, which discharge into the Grasse River, a weir test was out of the question. Before beginning

* Presented at the meeting of February 5th, 1913.

the tests, therefore, it became necessary to decide between the current meter and Pitot tube as instruments for measuring the flow.

In favor of the current meter is its simplicity and the facility with which it may be used in tail-races. Against it is the fact that, notwithstanding the large number of observers who have experimented with it and the large number of records which have been made by it, there is still a great deal of well-founded skeptical discussion as to the accuracy of the instrument when the records are based solely on the still-water rating.

The Pitot tube has been not only a subject of adverse criticism but also an instrument about which much positive ignorance seems to exist. It would seem, however, that the recent demonstrations of the accuracy of Darcy and Bazin's experiments, 1865, by Messrs. William Monroe White,* L. F. Moody, Gardner S. Williams, John R. Freeman, and others, should sufficiently dispel all doubts as to the reliability of the instrument. On the other hand, where the velocity of the water is variable from moment to moment, a large number of readings must be taken and reduced to a mean square root in order to obtain sufficiently close approximations to the average in any one velocity determination.

After a final summary of advantages and disadvantages, it was decided to use both the screw and cup types of current meter, and to check their records, if necessary, with a Pitot tube.

E. E. Haskell, M. Am. Soc. C. E., Director of the College of Civil Engineering, Cornell University, made up one of his screw type meters especially for the series of tests to be undertaken, and a new Gurley-Price meter of the cup type (No. 600) was purchased from the makers at Troy. Master Mechanic D. J. Jones, of the Power Company, made up a Pitot tube, following as nearly as possible the description of Tube *N* given in Mr. White's paper, mentioned above.

The following are the general conclusions concerning current meters which have been drawn from the tests with the foregoing equipment:

1.—†When a cup meter is run in perturbed water it will register

* "The Pitot Tube; Its Formula," by William Monroe White, *Journal*, Association of Engineering Societies, Vol. XXVII, 1901, p. 35.

† In a discussion of a paper on current meter and weir discharge comparisons by Edward C. Murphy, Assoc. M. Am. Soc. C. E., *Transactions*, Am Soc. C. E., Vol. XLVII, 1902, p. 379, Charles H. Miller, M. Am. Soc. C. E., describes certain experiments on screw and cup meters in the following language:

"The Haskell meter was lowered from one side of the skiff and one of the Price meters

a larger number of revolutions per second than a perfect still-water rating would indicate.

2.—†When a screw meter is run in perturbed water it will register a smaller number of revolutions per second than a perfect still-water rating would indicate.

3.—†In the foregoing sense, a cup meter is affected relatively to a much greater extent than a screw meter. In the tail-races at Massena, as an average, the cup meter was affected to the extent of 6%, while the Haskell meter was affected mostly by less than 1 per cent. In boilers of considerable violence the cup meter may easily over-register by 25%, while the screw meter will under-register by not more than 3 or 4 per cent.

4.—Either type of meter when run in perturbed water will give uniform records in equal times, provided these times are sufficiently long, the flow of the water itself being subject to an established regimen.

5.—If both types of meter are used simultaneously in perturbed water, the disparity between the discrepant velocities thus determined by the still-water rating may be taken as a basis for correcting the discrepant velocities.

6.—The average corrections thus obtained for the Haskell meter when run with the Gurley-Price meter in the tail-races at Massena varied from 0.5 to 0.9 of 1%, while the corresponding corrections for the Gurley-Price meter were about six times larger. Comparisons with the Pitot tube furnished substantially the same corrections as those obtained by comparison of the meters.

7.—It would seem to follow that current-meter observations based on still-water ratings without further correction should be made with great caution. On the other hand, it seems certain that the correction

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from the other side, both to the same depth (about 5 ft.). The registrations or revolutions of each, for periods of from 5 to 10 minutes, were noted; then, for similar periods of time, the skiff was caused to rock from one side to the other at short intervals, and the registrations recorded, the movement of the skiff giving to the meters a vertical motion of from 1 to 2 ft. Under this motion the revolutions of the Price meter increased, while those of the Haskell meter decreased, but in a less proportion than the increase of the Price meter, showing that the error due to instability of support is greater in the Price than in the Haskell meter. Lack of time prevented more extended observations, and in different velocities of current, which probably would have evolved some fixed difference due to the different construction of the meters in question."

Conclusions 1, 2, and 3, of the present paper seem to have been fairly stated by Mr. Miller, but as applied only to instability of support. They are here advanced as inherent characteristics of these different types of meter, and extended to apply, not only to cases of instability of support, but also to these current meters generally, however supported, in flowing water.

for a cup meter when run at a good meter station on an open river is not large, while the corresponding correction for a screw meter may be negligible.

Plate XXIII shows the results of a number of ratings of the Haskell meter. The general method was similar to that adopted by the United States Lake Survey, as described in the reports by F. C. Shenehon, M. Am. Soc. C. E.,* covering his current-meter observations. The meter was suspended either by cable or rod from a bowsprit at the head of a light skiff. The skiff, with an observer, was then towed back and forth over a 200-ft. base at a given velocity, one such double run constituting a velocity observation. The total time and total number of revolutions then stand for the transit of a 400-ft. base in perfectly still water.

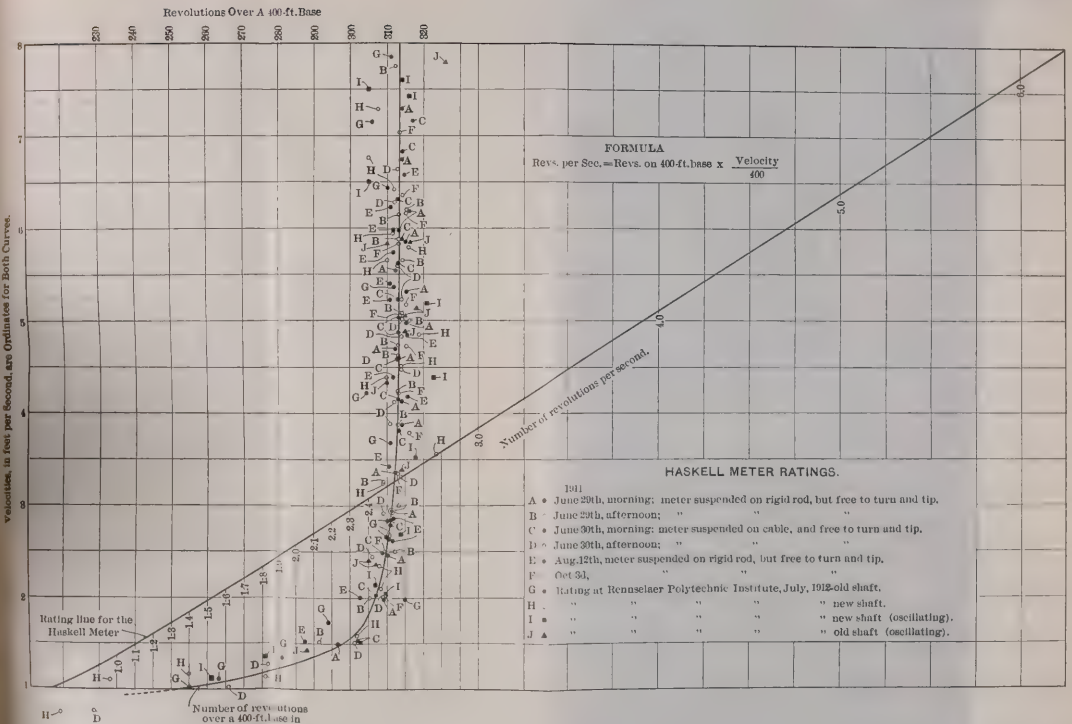
The first four ratings, on June 29th and 30th, 1911, were made along the longitudinal center line of Lock No. 15 in the old Cornwall Canal, at Cornwall, Ont. Both lock gates were open, but the north miter leaf of the lower gate, opening into the St. Lawrence River, was not fully into its recess in the north wall. The meter was about 4 ft. below the surface of the water, and the depth in the lock was 9 or 10 ft. The water in the lock had a tendency to surge at times. Sometimes there was considerable velocity through the lock, and at other times very little.

The ratings of August 12th and October 3d, 1911, were made in a bay of the Grasse River, opposite the power-house of the St. Lawrence River Power Company, at Massena, N. Y. The meter was about 2.25 ft. below the surface of the water, and the average depth under the base line was about 7 ft., with a shallow spot near the east end about 5 ft. deep. There was very little current, but at times there were surges due to the operation of the power-house just across the river.

The ratings of July, 1912, were made under the direction of Professor Arthur M. Greene, Jr., at the rating station of Rensselaer Polytechnic Institute, Troy, N. Y., after the meter had been used extensively at Massena and in North Carolina and Tennessee. The meter had been overhauled twice at Cornell University before these ratings, but long after those at Massena. The meter was also provided with a new shaft. After running a complete set of tests at Troy, the new shaft was put in and another set of tests was made.

* As described in Reports, Chief of Engineers, U. S. Army, 1900 to 1904, inclusive.

822A


Revolutions Over A 400-ft. Base
220
230
240
250
260
270
280
290
300
310
320
Velocities, in feet per Second, are Ordinates for Both Curves.
1
2
3
4
5
6
7
8
FORMULA
Revs. per Sec. = Revs. on 400-ft. base x Velocity / 400
Number of revolutions per second.
1.0
2.0
3.0
4.0
5.0
6.0
HASKELL METER RATINGS.
1911
A • June 29th, morning; meter suspended on rigid rod, but free to turn and tip.
B · June 29th, afternoon; " " "
C • June 30th, morning; meter suspended on cable, and free to turn and tip.
D ○ June 30th, afternoon; " " "
E • Aug. 12th, meter suspended on rigid rod, but free to turn and tip.
F · Oct 3d, " " "
G • Rating at Rensselaer Polytechnic Institute, July, 1912 old shaft.
H . " " " " " new shaft.
I ■ " " " " " new shaft (oscillating).
J ▲ " " " " " old shaft (oscillating).
Rating line for the Haskell Meter
Number of revolutions over a 400-ft. base in perfectly still water.

The curve of revolutions over a 400-ft. base was plotted from the first four ratings in the Cornwall Canal. It will be seen from Plate XXIII, however, that the subsequent ratings do not materially alter the position of this line, up to velocities of 5 or 6 ft. per sec. If the correction were made, the number of revolutions, as shown by the line, would be increased only ¼% at 4 ft. per sec., and reduced only ½% at about 8 ft. per sec. In fact, it may be seen that, out of about 100 double runs in the canal lock and Grasse River, not one plotted point differs by more than 1% from the line of revolutions as determined from the first four ratings for velocities of more than 1½ ft. per sec. For velocities of less than 1½ ft. per sec. the line of revolutions is so steep that errors of somewhat larger magnitude are to be expected. In the oscillating tests, the meter was oscillated parallel to the motion of the car, and covered an amplitude of 5¾ in. for every 8-ft. advance of the car. Transverse oscillations reduce the number of revolutions over a given base. The relations between velocity and revolutions over a 400-ft. base were then plotted, as shown on Plate XXIII, and from the resulting curve the rating line for still water was located and drawn, also as shown on that plate.

After having drawn the rate line, a reduction diagram, Plate XXIV, may be prepared, from which velocities may be read directly when referred to time and revolutions. This obviates either the necessity for reducing revolutions to revolutions per second, or of taking a fixed time for observing velocities, which is inexpedient where velocities are variant.

Plate XXV shows the Gurley-Price meter rating curves, obtained in a manner precisely similar to that used for the Haskell meter. In fact, the ratings were frequently made on the same day as for the Haskell meter.

In order that there may be no misunderstanding concerning these ratings, it may be stated that the tests were severe, when considered as a whole. Some of the ratings were made at a depth of 4 ft., some at a depth of 2 ft. 3 in., some in windy weather, some in calm weather, and several different observers acted at different times.

Under all these diverse conditions, it may be seen that the Haskell meter, in the Massena and Cornwall ratings, never varied by more than 1% in any individual observation, and that rating lines drawn for different ratings would differ by only 0.2 or 0.3 of 1% at the greatest.

The Gurley-Price meter exhibits a much greater range of variations, 5 or 6% up to velocities of 5 ft. per sec., which were considerably in excess of the mean velocities in the tail-races where the meter was used.

It is not contended here that a satisfactory still-water rating under perfect conditions cannot be obtained with a meter of this type; it is intended, however, to show that varying conditions of water produce relatively larger variations in the records of the cup meter than in those of the screw meter.

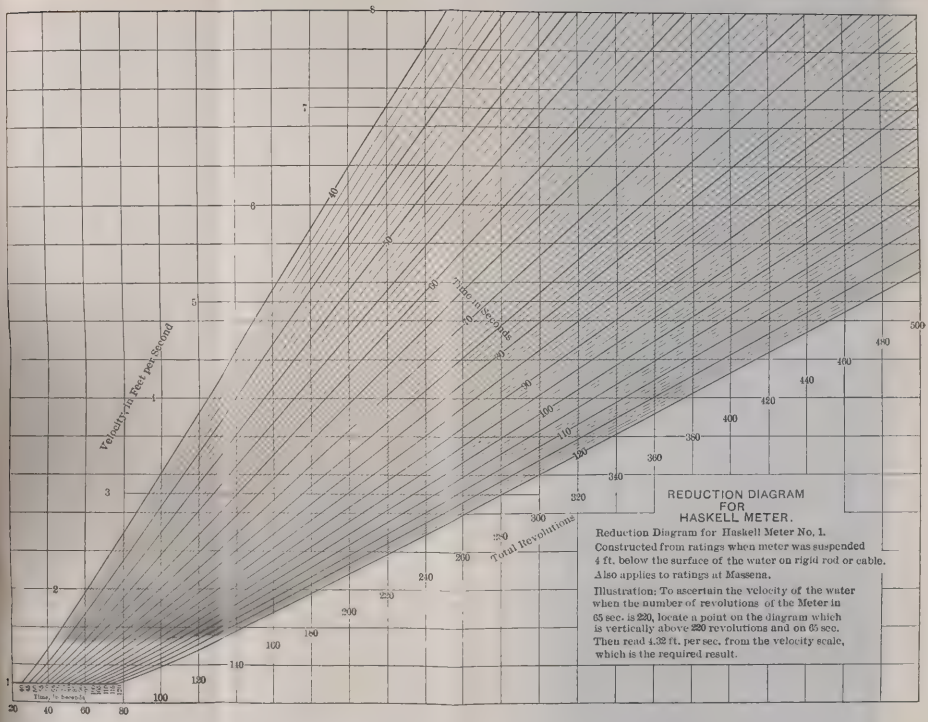
On comparing the maker's rating curve with the several ratings in the canal and in the bay of Grasse River, it will be seen that the number of revolutions of the Gurley-Price meter over a 400-ft. base is generally higher in the canal and bay than in the maker's rating. Hence it is fair to suspect that a perfect still-water rating line of a cup meter is a line of minimum number of revolutions for such a meter, and that, when the water is disturbed in any degree, the number of revolutions over a fixed base will always be increased, other things being equal.*

Similarly, the contrary suspicion may be entertained, that a perfect still-water rating line of a screw meter is a line of maximum number of revolutions for such a meter, and that, when the water is disturbed in any degree, the number of revolutions over a fixed base will always be decreased, other things being equal.

Since, under similar circumstances, the deviations of the Gurley-Price meter are about six times as large as those of the Haskell meter, but in the contrary sense, it would seem fair to conclude that when the meters are run simultaneously in flowing water, one-seventh of any difference in the velocities thus determined from the still-water ratings should be attributed to a deviation of the Haskell meter, while six-sevenths should be attributed to a simultaneous deviation of the Gurley-Price meter.

Thus, if the difference between the velocities determined by the Haskell and Gurley-Price meters should be 0.42 ft. per sec., 0.06 ft. per sec. should be added to the record of the Haskell meter, while 0.36 ft. per sec. should be deducted from the corresponding record of the Gurley-Price meter. In strictness, this correction should be

* Certain interesting experiments by Professor L. F. Moody, in the rating station at Rensselaer Polytechnic Institute, seem to agree with the writer's experiments on this point and confirm the general conclusion.



REDUCTION DIAGRAM FOR HASKELL METER.

Reduction Diagram for Haskell Meter No. 1.

Constructed from ratings when meter was suspended 4 ft. below the surface of the water on rigid rod or cable.

Also applies to ratings at Massena.

Illustration: To ascertain the velocity of the water when the number of revolutions of the Meter in 65 sec. is 220, locate a point on the diagram which is vertically above 220 revolutions and on 65 sec. Then read 4.32 ft. per sec. from the velocity scale, which is the required result.

applied to revolutions per second, but, as the number of revolutions is so nearly proportional to the velocity, and as the corrections are relatively small, no serious error results. The particular ratio here determined, of course, applies only to the two individual meters as used in the tail-races at Massena by the writer.

In order to establish more satisfactorily the principle mentioned above, the boat was rocked during several of the runs of a rating, imparting to the meter which was being rated an oscillation in an arc about 3 ft. long and about 2½ ft. in radius, and at right angles to the direction in which the meter was being drawn through the water. The result was that the Gurley meter was found to increase its number of revolutions by 15% while the Haskell meter reduced its number of revolutions by 2½%, practically the same ratio as that deduced above from the rating curves. The effect of longitudinal oscillations was tested also. The result of special tests of this character at Rensselaer Polytechnic Institute may be seen on Plate XXIII. Subsequently, other tests on a new Haskell meter showed that longitudinal oscillations of 22 in. in 6 ft. advance of car retarded the meter slightly, up to velocities of about 2 ft. per sec., above which the meter was accelerated, the error being less than 2% at 9 ft. per sec. The net result in perturbed water, however, seems to be slight retardation in all cases.

It is now desirable to show how these principles have been used to correct the current-meter records actually observed in the tail-races during the turbine tests referred to at the outset. In order to do this properly, it will be necessary to state another conclusion, based on the current-meter records, which the writer hopes to take up more fully in the future under another title. The conclusion is as follows:

Under an established regimen, the distribution of the flow of water in a tail-race obstructed by stilling racks is fixed in character, the ratio of the average velocity at any given point to that at any other given point in the race being constant. This proposition applies over a considerable range in the total amount of discharge, but supposes the actual wetted cross-section of the race to remain the same.

The foregoing proposition was proven from a number of the turbine tests by computing and compiling the relative velocities at the 84 meter points in the races where velocity observations were taken. A turbine unit discharged through two tail-races, each about 15 ft.

wide, while the depth of the water varied from 15 to 16 ft. The meter section of each race was divided into six equal vertical, and seven equal horizontal, strips, and the meter points were the forty-two intersections of the median lines of the strips. The verticals were numbered 1 to 6 from west to east, while the depths were numbered 1 to 7 downward.

To demonstrate the truth of the principle of a fixed distribution of flow in tail-races, and also as a verification of Conclusion No. 4, Tables 1 to 4 are given:

TABLE 1.—GURLEY-PRICE METER IN WEST RACE, SHOWING CONSTANCY OF VELOCITIES TAKEN IN QUICK SUCCESSION. VELOCITIES WERE OBSERVED WITH A SINGLE STOP-WATCH PROVIDED WITH TWO SECOND HANDS.

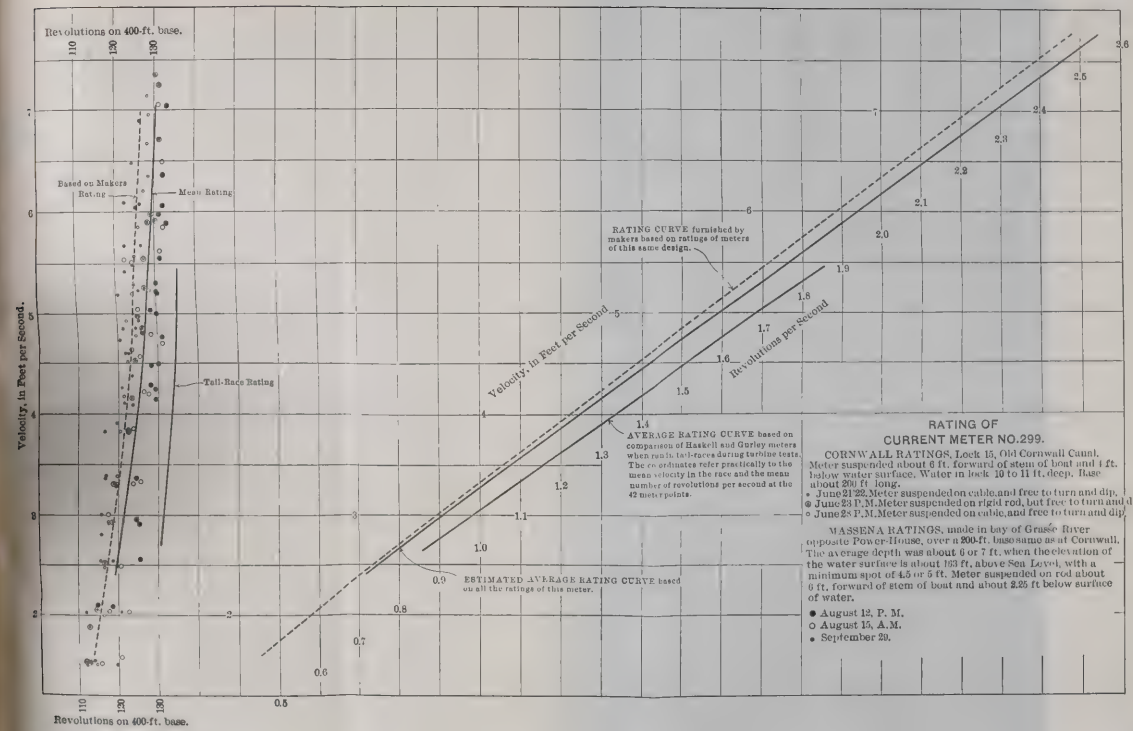
Test of September 9th, at 0.82 gate.
The numbers are of revolutions per second.

Depths.	Verticals.						Totals of lower set.
	1	2	3	4	5	6	
1	1.32	1.40	1.12	0.88	0.90	0.94	6.62
	1.36	1.42	1.16	0.89	0.87	0.92	
2	1.23	1.44	1.40	1.23	1.20	1.16	7.70
	1.24	1.44	1.40	1.24	1.22	1.16	
3	1.35	1.44	1.62	1.42	1.34	1.31	8.45
	1.35	1.44	1.59	1.41	1.35	1.31	
4	1.37	1.44	1.43	1.30	1.31	1.37	8.21
	1.36	1.44	1·43	1.30	1.32	1.36	
5	1.53	1.47	1.52	1.40	1.26	1.38	8.57
	1.49	1.48	1.53	1.42	1.27	1.38	
6	1.70	1.72	1.80	1.72	1.51	1.66	10.09
	1.69	1.72	1.80	1.72	1.50	1.66	
7	1.60	1.81	1.84	1.78	1.78	1.86	10.64
	1.58	1.80	1.84	1.78	1.79	1.85	
Totals of lower set.	10.07	10.74	10.75	9.76	9.32	9.64	60.28

The register was started first and the observer noted the pointer as it clicked from revolution to revolution of the meter. By synchronizing with the rythm of the register, it was possible for the observer to start the stop-watch almost exactly at the moment the register clicked to zero. Similarly, the primary second hand could be stopped after any desired number of revolutions while the secondary hand could be stopped at, say, twice that number. Thus two or more velocities in immediate succession could be determined for each meter point in the tail-races.

In support of the principle of fixed distribution, it may be stated that the percentages corresponding to the figures of Tables 1 and 2 at the 84 meter points agree remarkably well with the corresponding ones of Tables 3 and 4 after applying certain necessary small corrections. Tables 1 and 2 each represent an individual set of double discharge measurements in one of the races, while Tables 3 and 4 have

PLATE XXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1259.
GROAT ON
CUP AND SCREW CURRENT METERS.


Revolutions on 400-ft. base.
110
120
130
Based on Makers Rating
Mean Rating
Tail-Race Rating
Velocity, in Feet per Second.
RATING CURVE furnished by makers based on ratings of meters of this same design.
Velocity, in Feet per Second
Revolutions per Second
AVERAGE RATING CURVE based on comparison of Haskell and Gurley meters when run in tail-races during turbine tests. The co-ordinates refer practically to the mean velocity in the race and the mean number of revolutions per second at the 42 meter-points.
ESTIMATED AVERAGE RATING CURVE based on all the ratings of this meter.
Revolutions on 400-ft. base.

RATING OF CURRENT METER NO. 299.

CORNWALL RATINGS, Lock 15, Old Cornwall Canal. Meter suspended about 6 ft. forward of stem of boat and 1 ft. below water surface. Water in lock 10 to 11 ft. deep. Base about 200 ft. long.

- June 21-22. Meter suspended on cable, and free to turn and dip.
- ⊕ June 23 P.M. Meter suspended on rigid rod, but free to turn and dip.
- ◦ June 25 P.M. Meter suspended on cable, and free to turn and dip.

MASSENA RATINGS, made in bay of Grasse River opposite Power-House, over a 200-ft. base same as at Cornwall. The average depth was about 6 or 7 ft. when the elevation of the water surface is about 163 ft. above Sea Level, with a minimum spot of 4.5 or 5 ft. Meter suspended on rod about 6 ft. forward of stem of boat and about 2.25 ft. below surface of water.

- ● August 12, P. M.
- ○ August 15, A.M.
- • September 29.

TABLE 2.—Haskell Meter in East Race, Showing Constancy of Velocities Taken in Quick Succession. The Comparisons are Not as Good as with the Gurley-Price Meter, Because the Velocities were Taken with Two Different Watches, One of which was Not very Reliable.

Test of September 9th, at 0.82 gate.
The numbers are of feet per second.

Depths.	Verticals.						Totals of lower set.
	1	2	3	4	5	6	
1	3.23	2.74	2.48	2.41	2.94	3.75	17.45
	3.12	2.75	2.50	2.39	2.97	3.72	
2	3.62	....	....	....	....	....	21.49
	3.67	3.49	3.26	3.27	3.79	4.01	
3	4.12	3.96	3.53	3.91	4.13	4.09	24.24
	4.22	3.98	3.79	3.98	4.17	4.10	
4	4.21	3.92	3.64	3.73	3.88	4.07	23.48
	4.23	3.95	3.64	3.70	3.91	4.05	
5	4.30	3.90	3.73	3.68	3.61	4.25	23.57
	4.35	3.94	3.73	3.63	3.64	4.28	
6	4.77	4.25	4.74	4.61	4.18	4.44	27.25
	4.80	4.31	4.75	4.68	4.22	4.49	
7	5.07	5.27	5.32	5.05	4.98	4.61	30.32
	5.12	5.27	5.35	5.01	4.99	4.58	
Totals of lower set.	29.51	27.69	27.02	26.66	27.69	29.23	167.80

The two velocities at each meter point were taken in immediate succession in a manner similar to that described in the foot-note under Table 1, except that two stop-watches were used. The observations were about 1 min. in duration for the first velocity and twice as long for the second velocity.

TABLE 3.—Relative Distribution of Flow in West Race of Unit No. 3 During Tests of July and August. Water Surface at Elevation 162.85.

Depths.	Verticals.						Totals.
	1	2	3	4	5	6	
1	2.16	2.18	1.84	1.53	1.50	1.97	11.18
2	2.11	2.41	2.32	2.07	2.02	2.22	13.15
3	2.04	2.37	2.46	2.31	2.18	2.14	13.50
4	2.13	2.29	2.34	2.20	2.26	2.36	13.58
5	2.19	2.41	2.57	2.29	2.27	2.67	14.40
6	2.65	2.87	3.00	2.85	2.42	2.68	16.47
7	2.72	2.87	3.00	3.06	3.03	3.02	17.70
Totals	16.00	17.40	17.53	16.31	15.68	17.06	99.98

The numbers are percentages of the sum of the velocities at the 42 meter points in the race. Thus, in Vertical 1, Depth 1, the velocity is 2.16% of the sum of all the velocities at the meter points.

been compiled as average percentages from a number of such sets of observations.

In verification of Conclusion No. 4, it may be seen that each pair of velocities determined at the same point by either meter consists practically of two equal velocities. This truth has been found to apply throughout the turbine tests.

TABLE 4.—RELATIVE DISTRIBUTION OF FLOW IN EAST RACE OF UNIT NO. 3 DURING TESTS OF JULY AND AUGUST. WATER SURFACE AT ELEVATION 162.85.

Depths.	Verticals.						Totals.
	1	2	3	4	5	6	
1	1.72	1.69	1.66	1.85	2.16	2.24	11.30
2	2.02	2.04	2.00	2.15	2.24	2.24	12.70
3	2.42	2.31	2.19	2.20	2.31	2.29	13.70
4	2.62	2.54	2.26	2.12	2.17	2.45	14.20
5	2.59	2.41	2.18	2.16	2.13	2.43	13.90
6	2.73	2.49	2.80	2.82	2.59	2.72	16.20
7	3.00	3.04	3.11	3.02	2.97	2.88	18.00
Totals	17.12	16.55	16.21	16.33	16.57	17.22	100.00

The percentages of Tables 3 and 4 are computed by dividing the velocity at any meter point by the sum of the velocities at all the meter points.

Attention should be directed to the fact that while water discharged from a turbine will be much quieted by stilling racks, yet the latter themselves are sources of disturbance, and that the violent agitation caused by the turbines is merely supplanted by an entirely different type of disturbance produced by the racks. The effect of the racks is to render the flow parallel to the side-walls of the race, but the body of the water is full of small eddies and boilers of more or less violence. The parallelism of flow at each meter point was tested by a kind of inverted weather vane, which the writer understands was first used by John R. Freeman, M. Am. Soc. C. E. The indications of parallelism of flow by this vane were extremely satisfactory at all points.

The meter section was about 30 ft. down stream from the racks. The effect of the disturbance from the racks, however, was plainly indicated by the divergence of the velocity records, based on the still-water ratings, when the two meters were run in quick succession at any meter point. It is well to note here that the velocity was greater near the bottom of the race than at the top, and that the agitation

by the racks was greater near the top than at the bottom, where the flow was extremely steady and subject to little or no agitation. This is clearly shown by the records, and especially by the fact that the clicking of the register was almost absolutely uniform when the meters were near the bottom. It is also shown by the fact that there was always a greater disparity between the meter records near the surface than at the bottom.

To be precise, the corrections for the meters should be computed for each of the 84 meter points in the races. Practically, the corrections may be made for the average velocities in the seven horizontals, because the velocities along any horizontal do not vary widely.

An example of the computation of corrections for the Haskell and Gurley-Price meters is given in Table 5.

TABLE 5.—Computation of Velocity Corrections.

Horizontal in which meter was run.	Velocities Based on the Still-Water Ratings.		Difference.	One seventh of difference + Haskell = true velocity.	Velocity excessive by Gurley. Percentage.	Velocity deficient by Haskell. Percentage.
	Gurley.	Haskell.				
1	3.94	3.36	0.58	3.44	14.5	2.4
2	4.46	3.90	0.56	3.98	12.0	2.0
3	4.74	4.44	0.30	4.48	5.8	0.96
4	4.85	4.62	0.23	4.65	4.3	0.72
5	5.08	4.89	0.19	4.92	3.2	0.55
6	5.68	5.55	0.13	5.57	2.0	0.34
7	6.32	6.15	0.17	6.17	2.3	0.38
Totals...	35.07	32.91	2.16	33.21		

$$\text{Average error of Gurley meter} = \frac{6}{7} \times \frac{2.16}{33.21} = 5.6 \text{ per cent.}$$

$$\text{Average error of Haskell meter} = \frac{1}{7} \times \frac{2.16}{33.21} = 0.93 \text{ " "}$$

The method given in Table 5 for computing the corrections for the velocities is based solely on the current-meter ratings and the rocking experiments in conjunction with the fundamental propositions before stated. It is desirable, therefore, to have independent means of checking the methods and the numerical results.

Accordingly, a Pitot tube was operated at the meter points, in Depths 2, 3, 5, 6, and 7, simultaneously with the Haskell meter. While the meter was recording in Vertical 3 the Pitot was recording in Vertical 2. After the record was taken, both the meter and Pitot were

advanced to the next succeeding verticals, so that the meter would then record in Vertical 4 while the Pitot was in Vertical 3. Depths 1 and 4 were omitted because the tail of the Pitot was out of water in Depth 1, and the construction of the piping of the Pitot did not conform with the staging for the observers in such a way as to admit of readings in Depth 4. This might have been remedied, but no great importance was attached to these facts.

The results of these tests are given in Table 6.

It should be noticed in Table 6 that the velocities given in Verticals 1 and 6 were not taken in quick succession. The meter and tube were traversed across each horizontal, beginning with the Pitot tube in Vertical 1 and the meter in Vertical 2, and ending with the Pitot tube in Vertical 5 and the meter in Vertical 6. After the five horizontals were traversed in this manner, the Pitot tube and meter were interchanged in relative position, and readings were taken at the five depths with the Pitot tube in Vertical 6 and the meter in Vertical 1. All other readings by the meter or Pitot tube were followed immediately by a reading of the other instrument.

The writer does not wish to make too much of these Pitot tube tests. The experimenters were not familiar with the instrument, as they had used it on only one other occasion. The dynamic and static columns were drawn up on a scale graduated to single tenths and half tenths of a foot and the differences in head were merely estimated to the nearest hundredth of a foot. This may seem crude, but when it is considered that the water columns are both vibrating, sometimes violently, that twenty or thirty readings are taken for each velocity, and that all percentage errors are nearly split in two by the process of extracting the square root of the head, it will be seen that the method is probably as accurate as need be for the purpose at hand. Indeed, the results seem to justify this conclusion.

Attention may be directed to three general facts shown by the summary of Table 6. The ratios for the verticals increase toward the right in the west race. They increase toward the left in the east race. The ratios for the horizontals increase upward in both races. On the whole, the tendency is for the ratio of the Pitot velocity to the meter velocity to increase upward, as it should, as it has been shown that the meter is retarded toward the surface.

It is not wholly clear to the writer, however, why the Pitot tube

TABLE 6.—Comparisons Between Velocities Determined by Pitot Tube and Haskell Meter at Various Points in the Tail-Races of Unit No. 5.

Depth.		Verticals. 1		2		3		4		5		6		Totals.		Ratio.
		Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	
2	*A*	....	...	4.06	4.22	3.84	3.29	3.76	3.33	3.86	3.63	....	....	15.52	14.47	1.072
	A	....	....	4.15	4.09	3.86	3.11	3.49	3.38	3.65	3.22	....	....	15.15	13.80	1.098
	B	....	....	3.70	3.34	3.92	3.43	4.19	3.80	4.40	4.40	....	....	16.21	14.97	1.082
	B	....	....	3.84	3.54	3.83	3.77	4.32	4.30	4.40	4.48	....	....	16.39	16.09	1.020
	C	2.56	2.49	2.34	2.34	2.30	2.28	2.08	2.21	2.32	2.17	1.68	1.77	13.28	13.26	1.001
		2.56	2.49	18.09	17.53	17.75	15.88	17.84	17.02	18.63	17.90	1.68	1.77	76.55	72.59	1.055
3	*A*	....	....	4.22	4.21	4.14	4.02	4.05	3.98	4.35	4.20	....	....	16.76	16.41	1.020
	A	....	....	4.35	4.19	4.11	4.12	4.15	3.95	4.40	4.20	....	....	17.01	16.46	1.033
	B	....	....	4.06	4.10	4.29	4.44	4.31	4.36	4.46	4.53	....	....	17.12	17.43	0.983
	B	....	....	4.04	4.21	4.37	4.41	4 40	4.35	4.45	4.43	....	....	17.26	17.40	0.991
	C	2.58	2.71	2.36	2.39	2.34	2.41	2.15	2.21	2.36	2.50	2.09	2.06	13.88	14.28	0.972
		2.58	2.71	19.03	19.10	19.25	19.40	19.06	18.85	20.02	19.86	2.09	2.06	82.03	81.98	1.001
5	*A*	....	....	5.26	5.41	4.57	4.67	4.37	4.41	4.32	4.32	....	....	18.52	18.81	0.984
	A	....	....	5.27	5.37	4.66	4.89	4 48	4.47	4.33	4.39	....	....	18.74	19.12	0.980
	B	....	....	4.31	4.13	4.02	4.00	4.06	4.10	4.39	4.44	....	....	16.78	16.67	1.006
	B	....	....	4.43	4.14	4.08	3.95	4.11	4.00	4.34	4.33	....	....	16.96	16.42	1.032
	C	2.61	2.69	2.91	2.89	2.48	2.53	2.61	2.64	2.34	2.46	2.16	2.14	15.11	15.35	0.985
		2.61	2.69	22.18	21.94	19.81	20.04	19.63	19.62	19.72	19.94	2.16	2.14	86.11	86.37	0.998
6	*A*	....	....	5.36	5.61	5.47	5.55	5.41	5.55	4.93	4.91	....	....	21.17	21.62	0.978
	A	....	....	5.46	5.57	5.47	5.70	5.53	5.62	4.95	5.06	....	....	21.41	21.95	0.976
	C	2.74	2.80	2.98	3.07	3.07	3.10	2.82	2.89	2.60	2.57	2.32	2.17	16.53	16.60	0.996
		2.74	2.80	13.80	14.25	14.01	14.35	13.76	14.06	12.48	12.54	2.32	2.17	59.11	60.17	0.982

Remarks: *A*—tests in W. race at 0.7 gate; *B*—tests in E. race at 0.7 gate; *C*—tests in W. race at 0.37 gate.

TABLE 6.—(*Continued.*)

Depth.		1		2		3		4		5		6		Totals.		Ratio.
		Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	Pitot.	Meter.	
						Verticals.										
7.....	*A*	....	....	5.48	5.81	6.05	6.32	6.13	6.19	5.60	5.79	....	....	23.26	24.11	0.965
	A	....	....	5.56	5.79	6.16	6.32	6.21	6.03	5.66	5.79	....	....	23.59	23.93	0.985
	C	3.11	3.07	3.12	3.15	3.28	3.32	3.20	3.16	2.86	2.90	....	....	18.32	18.23	1.005
	C	3.03	3.07	3.06	3.20	3.24	3.33	3.24	3.19	2.99	3.00	2.75	2.63	15.56	15.79	0.985
		6.14	6.14	17.22	17.95	18.73	19.29	18.78	18.57	17.11	17.48	2.75	2.63	80.73	82.06	0.984
Totals..	*A*	....	....	24.38	25.26	24.07	23.85	23.72	23.46	23.06	22.85	....	....	95.23	95.45	0.998
	A	....	....	24.79	25.01	24.26	24.14	23.86	23.45	22.99	22.66	....	....	95.90	95.26	1.008
	B	....	....	12.07	11.57	12.23	11.87	12.56	12.26	13.25	13.37	....	....	50.11	49.07	1.021
	B	....	....	12.31	11.89	12.28	12.13	12.83	12.65	13.19	13.24	....	....	50.61	49.91	1.014
	C	16.63	16.83	16.77	17.04	16.71	16.97	16.10	16.30	15.47	15.60	11.00	10.77	92.68	93.51	0.991
		16.63	16.83	90.32	90.77	89.55	88.96	89.07	88.12	87.96	87.72	11.00	10.77	384.53	383.17	1.004
Ratio...	*A*	.....		0.965		1.010		1.010		1.010		.....		.....		
	A	.....		0.990		1.006		1.018		1.013		.....		.....		
	B	.....		1.042		1.030		1.025		0.991		.....		.....		
	B	.....		1.036		1.012		1.015		0.995		.....		.....		
	C	.....		0.985		0.985		0.988		0.993		.....		.....		
		0.988		0.995		1.007		1.011		1.003		1.021		1.004		

Remarks: *A*—tests in W. race at 0.7 gate; *B*—tests in E. race at 0.7 gate; *C*—tests in W. race at 0.37 gate.

velocity should be in the neighborhood of 1½% less than the meter velocity at the greater depths. It is not improbable that this would have been partly compensated for, had the observations in the east race been continued to Depths 5 and 6, but probably not entirely. In fact, there seems to be a reciprocal relation, as noted above, between the readings in the two races. This would follow naturally, owing to the symmetry of the flow in the races as to the central dividing wall, and the fact that the meters always rotate in the same direction. Thus, if there is an eddy clockwise at a certain position in either race, there should be a symmetrically located eddy counter-clockwise in the other race, and these might have opposite effects on the meter, the Pitot tube not being affected in the same way.

It is not impossible that the Haskell meter was running a trifle faster near the bottom of the race than the actual still-water ratings would indicate that it should. This would result from the fact that the rating boat was not a rigid support, nor was the water absolutely still during the ratings. This would necessitate shifting the rate line of the Haskell meter toward its ideal position of a maximum number of revolutions. This, in turn, would diminish all discharge records based on still-water ratings. On the other hand, to be consistent, it would follow that the rate line of the Gurley-Price meter would have to be shifted toward the maker's rate line, which would make for a larger correction to be added to velocities by the Haskell meter when based on still-water ratings, thus leaving true velocities in Table 5 practically unchanged. In corresponding manner, all the ratios of Tables 6 and 7 would be increased, while the meter velocities would be less in the same proportion, the effect of which would be to leave the corrected true velocities corresponding to the tables unaltered, they being in fact those of the Pitot tube. Thus, discharge estimates would not be affected materially in either case. The most reasonable explanation, however, is that the Pitot tube itself is variously affected by small amounts in different parts of the raceways.

The object, however, is not to speculate on such matters, but simply to check the computation of a small correction to the velocity, which correction may be in error by 50% and yet not affect the discharge measurements by the meter by more than three or four tenths of 1 per cent.

Interpolating a percentage for Depth 1, which was not observed in

the tests, equal to the mean given for Depth 2, 1.055, and another in Depth 4, taking an average between the means for Depths 3 and 5, practically 1.000, there may be arranged the figures in Table 7.

TABLE 7.—AVERAGE CORRECTION FOR HASKELL METER, BASED ON COMPARISON OF CURRENT METER AND PITOT TUBE RECORDS.

Depths.	Ratio of velocity by Pitot tube to that by current meter, from Table 6.	Relative velocity in each horizontal, being a mean for Haskell meter.	Corrected relative meter velocity.
1.....	*1.055	10.20	10.76
2.....	1.055	11.86	12.51
3.....	1.001	13.50	13.51
4.....	*1.000	14.04	14.04
5.....	0.998	14.86	14.84
6.....	0.982	16.86	16.56
7.....	0.984	18.68	18.39
		100.00	100.61

* Interpolated velocity ratio.

It follows from Table 7 that an approximate correction of 0.61% should be applied to discharges determined by the Haskell meter when used in these tail-races, and basing velocities on the still-water rating.

As a matter of fact, no direct use whatever was made of the still-water rating curves of the Gurley-Price meter, except to compute the corrections for the meters, which seem to be substantially verified by the Pitot tube tests. In reality, the Gurley-Price meter was rated relatively to the Haskell by comparisons of the simultaneous records taken by the meters in the two races during the numerous turbine tests.

While the Haskell meter runs in the west race, the Gurley-Price meter runs in the east race, and *vice versa*. If a pair of turbine tests be selected where the conditions are substantially the same, the Haskell meter being in the west race during one of the tests and in the east race during the other test, the total sum of the 84 velocities by the Haskell meter may be plotted to the total sum of the 84 numbers of revolutions per second by the Gurley-Price meter. If a number of such points be plotted, the curve drawn through them will represent the functional relation between the sum of the 84 velocities by the Haskell meter and the corresponding total sum of the numbers of revolutions per second by the Gurley-Price meter at the 84 meter points in the two races.

PLATE XXVI.
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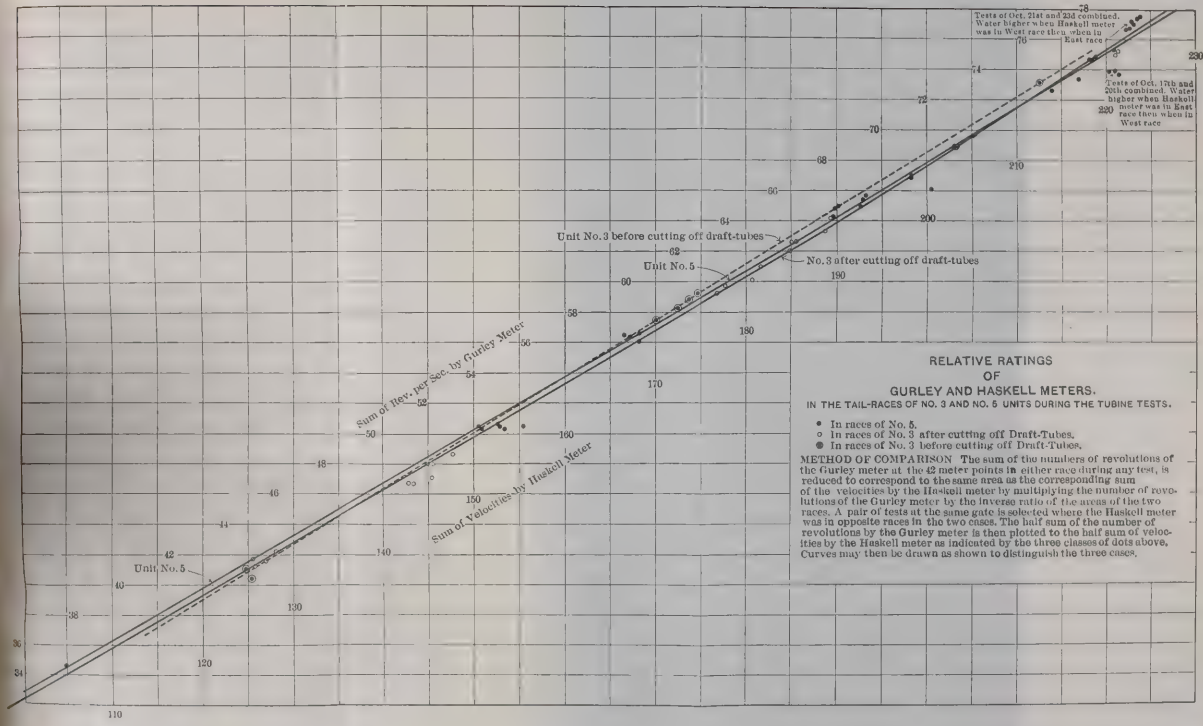
GROAT ON
CUP AND SCREW CURRENT METERS.


Tests of Oct. 21st and 23d combined. Water higher when Haskell meter was in West race than when in East race
Tests of Oct. 17th and 20th combined. Water higher when Haskell meter was in East race than when in West race
Unit No. 3 before cutting off draft-tubes
No. 3 after cutting off draft-tubes
Unit No. 5
Unit No. 5
Sum of Rev. per Sec. by Gurley Meter
Sum of Velocities by Haskell Meter
RELATIVE RATINGS
OF
GURLEY AND HASKELL METERS.
IN THE TAIL-RACES OF NO. 3 AND NO. 5 UNITS DURING THE TUBINE TESTS.
• In races of No. 5.
○ In races of No. 3 after cutting off Draft-Tubes.
● In races of No. 3 before cutting off Draft-Tubes.
METHOD OF COMPARISON The sum of the numbers of revolutions of the Gurley meter at the 42 meter points in either race during any test, is reduced to correspond to the same area as the corresponding sum of the velocities by the Haskell meter by multiplying the number of revolutions of the Gurley meter by the inverse ratio of the areas of the two races. A pair of tests at the same gate is selected where the Haskell meter was in opposite races in the two cases. The half sum of the number of revolutions by the Gurley meter is then plotted to the half sum of velocities by the Haskell meter as indicated by the three classes of dots above. Curves may then be drawn as shown to distinguish the three cases.

As the two races are so nearly alike, it may be accepted that, on the average, half the total sum of the 84 velocities will correspond to half the total number of revolutions per second at the 84 meter points, as a functional relation between the total sum of the velocities at the 42 meter points and the corresponding total sum of the revolutions per second at these points for either race taken separately.

Plate XXVI is such a relative rating curve, compiled from selected turbine tests, and may be used to reduce the records of the Gurley-Price meter to what they would have been by the Haskell meter. Thus, if in any tests on Unit No. 5 the Gurley-Price meter makes a total sum of revolutions per second of 50, then, by this curve, the Haskell meter would have made a record indicating a sum of 149.3 ft. per sec.

It was not found necessary to have any regard for the individual velocity by the meter at any point, except as a part of the total sum. A system of constants and ratios was devised, so that, to obtain the mean velocity in a raceway during any tests, it was merely necessary to multiply the sum of the velocities at the 42 meter points by a factor obtained from a curve. This, however, is a matter which the writer hopes to take up more in detail at another time, and therefore, he will confine himself here to the performance of the meters in the tail-races.

With this relative rating curve and the percentages of Tables 3 and 4, it would be quite possible to determine the absolute rating curve of either meter at each of the 84 meter points; but this is hardly necessary. It will suffice here, as a matter of interest, simply to conclude this paper by determining what the average absolute rating curve of the Gurley-Price meter was in the turbine tests. This may be done by dividing the numerical value of the abscissas and ordinates of Plate XXVI by 42 and plotting the resulting pairs of reduced coordinates to the corresponding reference lines in Plate XXV.

It may there be seen that the number of revolutions per second by the Gurley-Price meter in the tail-races was, on the average, about 6% greater than the still-water rating would indicate. In other words, if the writer had relied on this meter alone, without any other guide than the still-water rating curve, his discharge results would undoubtedly have been 6% too high.

These conclusions are not to be taken as casting any reflections on the Gurley-Price meter, which according to the writer's experience, is an admirably constructed instrument. In fact, the results could not have been obtained without it. The rating of the meter was perfectly definite at each of the 84 meter points in the races, but could not be determined directly by the still-water rating, which, in reality, is the only thing at fault. The still-water rating should never be applied without regard to the probable deviations of the speed of revolution due to disturbances in the water. An excellent check on the meters is to rate them where operated with a Pitot tube.

Addenda.

Since the foregoing was written, the writer has had a very instructive experience with several types of meter used simultaneously on a complicated network of mountain streams. The results of oscillating the meters have confirmed the general conclusion that in all cases, relatively, cups are accelerated considerably, while screws are retarded slightly, in turbulent water. The errors of the cup meter, based on still-water ratings, were from 3 to 6 times greater than those of the screw, and in the contrary sense.

Design of Meters.—The principal desideratum of a current meter is that it should give the resolved component of velocity in a direction fixed relatively to the meter. The fact that a cup over-registers in turbulent water while a screw under-registers, gives a basis of design which may be used to produce a meter possessing this characteristic more or less rigidly. If the blades of a screw are "cupped" to the proper extent, and in the right sense, the effect may thus be toward neutralizing the retardation which the screw would otherwise suffer.

There are other ways of producing such a meter. Professors Greene and Moody, of Rensselaer Polytechnic Institute, have conducted a series of experiments there which resulted in the production of a meter practically giving only the resolved components of velocity.

A conclusion to be drawn from these experiences is that a tail, or rudder, is a useless appendage to a meter used in stream gauging. The meter should be held rigidly in the stream or conduit, giving only the component of velocity perpendicular to the cross-section. A cup meter would give better results in turbulent water if it had no tail, but was simply allowed to run at the end of a vertical rod like an inverted cup anemometer.

The recording device for a meter designed to be held rigidly in the current should register positively for down-stream current and negatively for up-stream current. There are cases where the current beneath the surface is up-stream; and the old forms of meter make no distinction, as they should.

Accuracy of Velocity Determinations by Current Meters.—Under given conditions, the current meter is a very accurate instrument. To prove this, it is only necessary to take a series of observations at a given meter point, or better, at a given set of meter points, when conditions at the gauging section are constant. Repetitions of these observations will give results checking within 1 per cent.

It is not even necessary to take observations under constant conditions, as is shown by the writer's plottings of the sums of the metered velocities at selected sets of meter points at various gauging stations. These curves are drawn through the plottings of points the co-ordinates of which are the elevation of the water surface at the given gauging station and the sum of the velocities at a selected number (25 to 100) of meter points in the cross-section. In shallow streams the meter points should be at mid-depth.

Even in the case of shallow, turbulent, stony streams, such plotted points all lie within 1% of the finally determined curve, thus showing that the performance of the meter, be it of screw or cup type, is uniform and reliable.

The main sources of error are in the application of the still-water rating, the determination of the section area, and the determination of the distribution of flow through the cross-section. The errors due to an erroneous determination of the distribution of velocities are frequently styled the "errors due to the method of calculation."

Distribution Factor.—If the discharge and section area are determined accurately for any one gauging, the mean velocity in the section may be divided by the sum of the velocities at a selected set of meter points. This ratio may be called the distribution factor for the corresponding stage of the water. In this way, corresponding values of the distribution factor and stage of the river may be determined and plotted, the co-ordinates being the elevation of the water surface and corresponding value of the distribution factor. If the work has been good, the points will plot as close to a smooth curve as the plotting of points representing the sums of velocities at the selected set of meter points.

Section Area.—The section area, next to the error in applying the still-water rating, is probably the most uncertain element of a discharge determination, where the stream is shallow and stony. Nevertheless, it has been ascertained that the surface profile of such streams at reasonably good stations is a function of the stage of the water, in many cases being simply a horizontal line. By taking a minutely accurate profile of the bottom of such a stream along the cross-section, and a number of surface profiles at various stages, the section area may be plotted to the corresponding elevation of water surface. This is another element for the accurate determination of a discharge curve.

Statistical Synthesis of a Discharge Curve by Means of its Generating Elements.—According to the definitions given above, there are three principal generating elements of a discharge curve, all of which may be exhibited as curves plotted to the elevation of the water surface at the cross-section. These generating elements are, respectively, the sum of the velocities at a selected set of meter points, the distribution factor, and the area of cross-section.

If, for any given stage of water, the corresponding values taken from the curve of sums of velocities and the curve of distribution factors be multiplied together, the result is the mean velocity in the section for that stage. The product of this mean velocity and the section area taken from the area curve for the same stage is the discharge for that stage. In this way the discharges of the stream may be computed for all stages and plotted as a finally determined discharge curve.

The method is as strictly statistical as though the laws of probability had been applied formally, and the result is practically as satisfactory and infinitely easier to obtain. Moreover, errors are immediately detected and located.

Good work on the mountain streams of North Carolina and Tennessee has shown that individual discharge determinations based on the foregoing processes will plot within 1 or 2%, as extreme errors, of the final discharge curve.

Distribution of Flow.—The distribution of flow is determined in the ordinary manner by vertical velocity curves. It is best, however, to be systematic. A good system is to determine the ratio of the mid-depth velocity to the mean velocity in each vertical for the whole range

8-78A.

PLATE XXVII.
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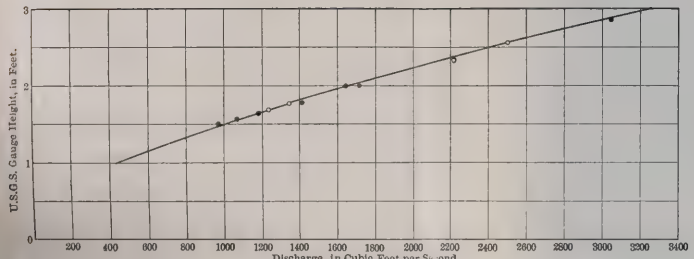
GROAT ON
CUP AND SCREW CURRENT METERS.

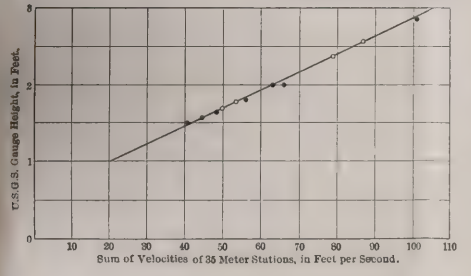


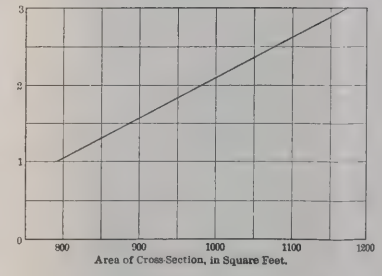
DISCHARGE CURVE FOR TUCKASEGEE RIVER AT BRYSON, N. C.

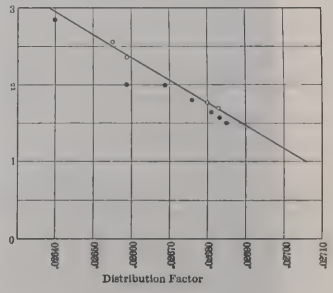
- • Observations with Gurley Meter
- ○ Observations with Haskell Meter

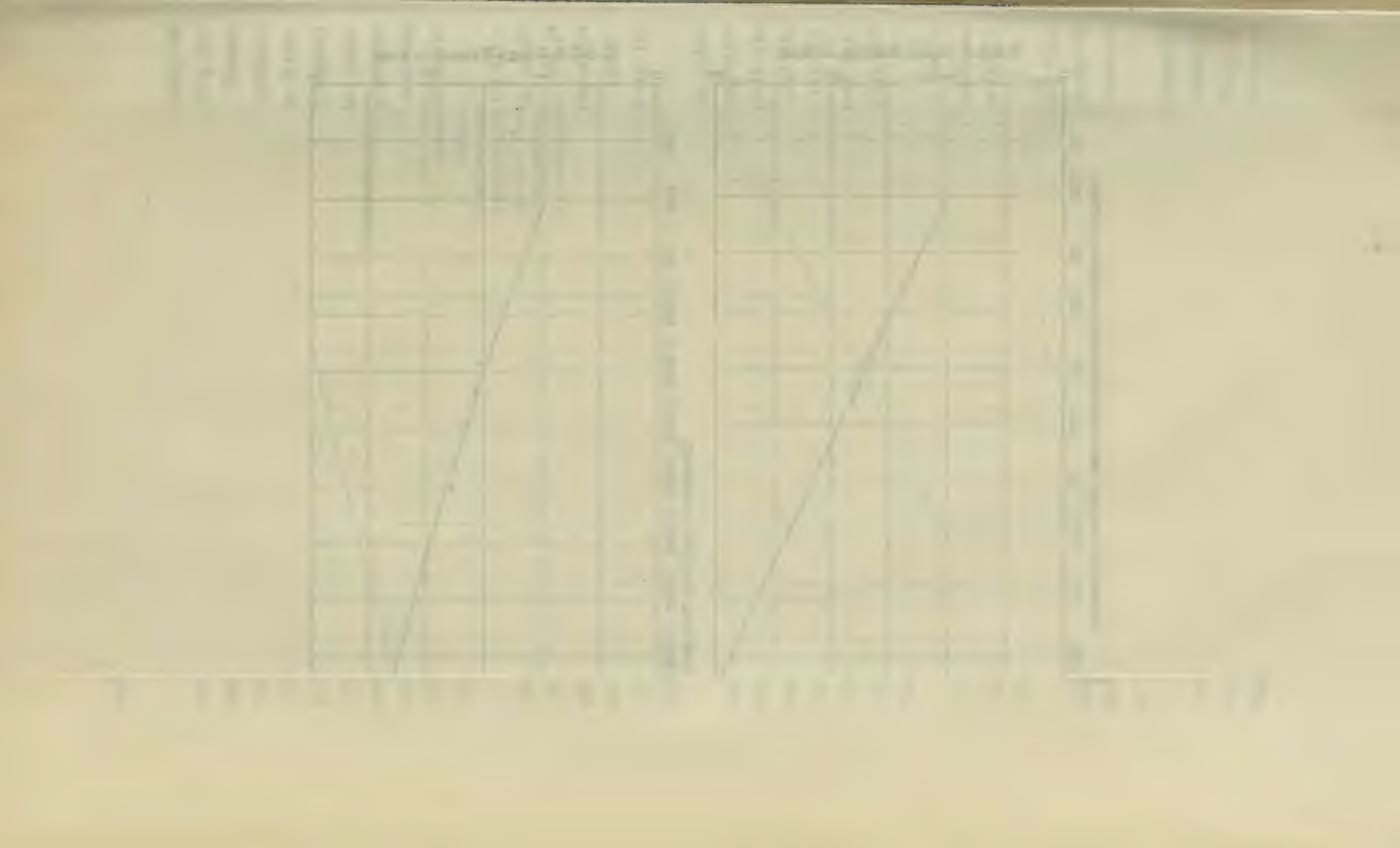
The discharge curve has been computed from the elementary curves obtained from observations with the Haskell Meter.

The points spotted on the discharge curve indicate the individual discharge obtained from each gauging.









of stages of the river. The writer has used an average value of these ratios for each stage of the river, by which the mean mid-depth velocity for the corresponding stage is to be divided, in order to obtain the mean velocity in the section before computing a discharge. In practice, a curve may be drawn giving the mean value of the ratio for each stage of the river. The determination of this ratio is a different thing from a gauging for discharge.

Discharge Gauging.—As described above, the writer prefers to take gaugings of shallow rivers at mid-depth at equidistant stations along the cross-section. The individual discharge is then computed by dividing the sum of the products of the corrected mid-depth velocities and their corresponding areas by the average value of the before-mentioned ratio for the proper stage. The discharges at the ends of the section, below obstructions in the stream and around and through piers of bridges, should be treated separately under the title "end discharges" or "additional discharges." This additional discharge may be plotted as a function of the stage of the river. Such a plotting shows whether the calculation has been made on a consistent basis by forming a well-determined curve. Otherwise, the computer has varied his method of attack inconsistently.

Error in Using Average Ratios.—Undoubtedly, an error is made in using an average ratio of any sort, unless that average be determined properly. Thus a time average and a space average of a quantity are two entirely different things. It is important, therefore, to use the correct form of averages. However, if the values of the ratio of mid-depth velocity to mean velocity in thirty or forty verticals at a station of fairly uniform depth and velocity all lie between 1.10 and 1.18 as extremes, with an arithmetical average of about 1.14, no serious error can result in using such an average value. In fact, in this particular case, a detailed computation and one based on the average value of 1.14 differed by only one-seventh of 1 per cent.

Room for Research.—In the writer's opinion, it would pay scientific investigators to develop a meter which under all conditions would give the resolved component of velocity in a given direction, rather than to attempt to determine any functional relation between the deviations of cup and screw meters from their still-water ratings. In the foregoing studies of the results at Massena it was thought sufficiently accurate to consider the error of the cup meter about six

times the error, in the contrary sense, of the screw meter. This ratio is undoubtedly a variable, but no serious error can result by assuming it to be constant where the extremes observed would make less than 1% difference in the discharge, as was the case in the turbine tests at Massena.

Practical Application of the Methods.—Table 8 exhibits the results of a number of discharge determinations at a station where the turbulence of flow is at a minimum. In fact, the disparity between the records of the Haskell and Gurley meters is less in these observations than for those at any of the other stations operated in North Carolina and Tennessee. The average ratio of mid-depth velocity to mean velocity in the vertical, for 45 vertical velocity curves, is 1.055. The curve has not been drawn.

TABLE 8.—Results of a Number of Discharge Observations Taken at the Station at the Wagon Bridge, Bryson, N. C.

Meter.	Gauge height.	*Sum of velocities in 35 verticals.	Section area.	Distribution factor.	Discharge.	Mean velocity.
Gurley	1.50	40.59	885.0	0.02685	965	1.090
Gurley	1.57	44.31	898.3	0.02684	1 068	1.189
Gurley	1.64	48.19	911.7	0.02681	1 179	1.292
Haskell	1.69	49.83	921.2	0.02683	1 230	1.337
Haskell	1.77	53.28	936.4	0.02680	1 338	1.429
Gurley	1.80	55.97	942.1	0.02676	1 410	1.498
Gurley	2.00	65.73	980.2	0.02659	1 713	1.748
Gurley	2.00	62.94	980.2	0.02669	1 646	1.680
Haskell	2.37	78.96	1 050.7	0.02659	2 208	2.100
Haskell	2.56	86.66	1 087.3	0.02655	2 502	2.301
Gurley	2.85	100.73	1 144.3	0.02640	3 044	2.660

*The velocities are here based solely on the still-water ratings of the meters.

Plate XXVII shows the plottings of the main elements of the discharge observations of Table 8, according to the statistical methods described herein. The discharge curve is determined by the three elementary curves, as already explained. The elementary curves are plotted from the records of the Haskell meter.

DISCUSSION

W. G. Price, M. Am. Soc. C. E. (by letter).—This paper gives the results obtained from a very large number of observations, and should be of much value. Before accepting these results as perfectly reliable, the writer would be pleased to know if the meters, in both the rating and stream tests, were suspended in precisely the same way and by supports of the same size? Mr. Price.

Were weights used in both cases; and were they always the same distance from the wheel?

Was the skiff so far from the wheel that it did not push the water ahead of itself a trifle and thus reduce the rate of revolution of the wheel? In some cases, a steamer ascending the Upper Mississippi will check slightly the flow of the water some hundreds of feet ahead of itself.

The writer has had an extensive experience with current meters during a period of seventeen years, beginning with 1879, during which time he measured the flow in the Mississippi, Missouri, and Ohio Rivers, and their large and small tributaries, many times, and in many places. The following propositions are the result of his experience:

The thread of a perturbed current, of course, is longer than that of a steady current. A meter of the cup type, without a vane, measures the thread of the horizontal current perturbations, but when provided with a vane, it measures a trifle more than the thread of the horizontal perturbed current. This is due to the effect on the vane of the perturbed current which swings the wheel transversely to the current. If a cup meter with no vane is fixed on a vertical rod, the current perturbations moving up and down in the vertical plane will decrease the revolutions of the wheel and thus offset the increase due to the horizontal perturbations. This action is due to the fact that the top and bottom faces of the meter wheel present a propeller-shaped surface to the water, and when vertical currents strike the wheel they tend to turn it in the opposite direction from that produced by the water flowing against the cups. For this reason the vane should not be used in streams shallow enough to permit of it being omitted, if there are perturbed currents. Larger streams, when the gauging station is carefully selected, are not perturbed to such an extent as to cause a serious error in meter measurements.

A meter of the propeller type will measure less than the thread of the perturbed current, because of the inability of the vane to keep the face of the wheel at all times precisely normal to the current. A current striking a propeller wheel sidewise does not increase its revolutions, as is the case with the cup meter. Each type of meter, in

Mr. Price. clear, straight, running water, will give the same accurate results, provided it has been accurately rated and is supported in the stream in precisely the same way as when rated, or by a method which does not produce an increase or decrease of current at the meter wheel.

When a weight is attached to the supporting rod under the meter, it reduces the discharge area and increases the velocity of the water in its vicinity, and thus increases the rate of revolution of the meter wheel. If the weight is placed 6 in. below the meter, the wheel, in each type, will run much faster than when the weight is 12 in. below the meter. A difference in the location of the weight, or the use of the weight in one case and not in the other, will produce all the difference, noted by the author for the Price meter, between ratings and stream measurements.

The propeller type of meter is often made to run more slowly between rating and stream measurements by a slight quantity of grit blowing into the bearings, or being carried in by the water. This type of meter is often made to run slow by a particle of dead grass or leaf (found moving in most streams) catching on the propeller. The propeller type of meter does not free itself from such débris, as does the cup meter.

When meters are rated in still water, it is necessary to wait 10 min. or more after each passage of the meter for the water to become still again. If the wind is blowing against the surface of the water, variable and sometimes rotary currents will be produced, which will make the rating inaccurate. The surface currents will run in one direction and the under current in the opposite direction. For this reason, the meter should be rated at a depth of more than 2½ ft., and in water more than 5 ft. deep.

A rating taken in running water is likely to be the most accurate, because the running water carries away the disturbing currents produced by the movement of the meter.

When a cup meter is accurately rated in still water and in ordinarily steady running water, the two ratings will be the same within a small fraction of 1%, and the rating curve can be drawn through all the plotted points obtained by the rating, so that no point will vary in revolutions as much as 1% from the curve.

The writer has used a Morse paper tape register and electric device to start and stop the register, or mark the paper tape, at the instant of passing the cross-wires suspended over the 200-ft. base, so that the fractions of revolutions of the cup meter wheel were determined at the beginning and end of the run; and such a rating in still water, with almost no wind, located all the plotted points within a very small fraction of 1% of the number of revolutions from the rating curve as drawn.

Mr. Price.

The ratings of both the Haskell and Price meters given on Plates XXIII and XXV are inaccurate, as the plotted points indicating the number of revolutions vary entirely too much. The errors in the Price ratings are apparently much greater than in the Haskell ratings.

Mr. Groat has conducted a series of observations in one tail-race where the currents were quite divergent, and the writer submits that the result of his experiment, treating as it does of one special condition, cannot be taken, either directly or by inference, to apply to general conditions of measurement of the flow of streams.

Under such peculiar conditions, where there are cross-currents, as in the Massena experiment, it is possible that the propeller type, owing to its peculiar construction, is less affected by such currents than the cup type; but if the propeller type were better for the measurement of streams where accurate measurements are impossible, owing to cross-currents, this fact would not indicate that such meters are better for general use.

A perfect still-water rating, of course, is what is required when the instrument is to be used in the measurement of straight, flowing streams. The usual rating in running water is made only to secure the same result as would be obtained by a perfect still-water rating. The small variations of the current from a straight line, as seen at the gauging section in most streams, makes only a negligible error in meter measurements.

A meter measures approximately the length of the curved thread of the current; therefore, if the thread is very divergent, inaccurate results will be obtained with any meter.

A gauging section having a perturbed current which would cause an error of 6% would give evidence to the eye by the boiling of the water, and would make the experienced observer hunt for a better section. However, better sections cannot possibly always be found.

A gauging section which has an up-stream current should never be used, and one which has very divergent currents cannot give accurate results with any method of measurement.

The first meter used by the writer had a cup wheel which revolved in a horizontal plane. It was designed by Gen. Ellis, and had been used by him in the Connecticut River. This meter did good work in water which was absolutely free from grit. Floating organic matter, such as leaves and grass, did not often affect it, as the turning of the wheel caused such débris to free itself. When the attempt was made to use it in the muddy water of the rising Ohio in 1882, it gave very inaccurate results, owing to grit getting into the bearings, which made its rate variable. The writer was then furnished with two very finely constructed meters, designed by Clemens Herschel, M. Am. Soc. C. E., and made by Buff and Berger especially for this work. Each was of

Mr. Price. the propeller type, with horizontal shaft, having agate bearings, and each was provided with a vane to hold the meter in line with the current in both the vertical and horizontal planes. They were as perfect instruments of the propeller type as the writer has seen.

These meters were tested by rating them in clear water, and gave good results. They were then rated in a bayou of roily water, an overflow from the Ohio, and the results were very inaccurate, owing to grit getting into the bearings, and leaves and other débris catching on the wheel and continuing to revolve with it. They were tested in the turbid Ohio River with no better results.

It was absolutely necessary to have a meter which would do accurate work, if the great Ohio flood of 1882, which was then on its way, was to be accurately measured. In designing and constructing what is called the Price meter, on this occasion, the cup type of wheel was adopted, for three reasons: The friction of the bearings is so small that this wheel can measure sufficiently low velocities, which is not the case with the propeller type; the revolving wheel will clear itself of débris; and, by using a vertical shaft, with inverted cup bearings filled with air and oil which will stay in by capillary attraction, no water with its load of grit can enter. The friction of the bearings thus becomes a constant quantity. The well-known and successful operation of these meters is due to these three features, which the writer believes cannot be secured by any other design.

The propeller type of meter, designed by Mr. Herschel, was in service before the Price meter, but has had quite a limited use, while several thousand Price meters have gone into service. This test of time, under all kinds of conditions, indicates that the cup meter is the most efficient for general purposes.

The writer believes that with an accurately rated cup meter of the type having no vane—which is much the better design for shallow, perturbed streams—more accurate results could have been secured at Massena.

Mr. Haskell. E. E. Haskell, M. Am. Soc. C. E. (by letter).—The writer has been greatly interested in the results set forth in Mr. Groat's paper, for he has never had occasion to use current meters under just such conditions.

The current meter was designed originally to measure velocities in open channels. It has developed into two distinct types: those having screw wheels and those having cup wheels. The first current meter, "Woltman's Mill", more than 100 years old, was of the screw wheel type. This meter has undergone various changes in form and in the mounting of its wheel, and, on the whole, has proved to be very accurate and reliable. The origin of the cup wheel meter is not as definite. It has undergone various changes, and the cups have been of various forms, namely, cones, hemispheres, and combinations of cones and

hemispheres. Although a great deal of good work has been done with meters of this type, they have not shown themselves to be anywhere near as reliable as those of the screw wheel type. In the writer's judgment, this is due to the fact that they do not always present the same area and form of wheel to the current pressure. The tilting of the meter, even through small angles, changes these, and consequently changes the rating curve or equation. In current meters of the screw wheel type, such tilting through small angles makes but a slight change in the area and form of wheel presented to the current pressure, and, accordingly, they follow much more closely their rating curve or equation. Mr. Haskell.

In the last paragraph on page 836, the author states, as follows:

"A conclusion to be drawn from these experiences is that a tail, or rudder, is a useless appendage to a meter used in stream gauging."

The writer's experience will not allow him to accept this conclusion. In a large river, the difficulty of holding the axis of the meter perpendicular to the cross-section becomes so great that it is out of the question. In large streams it is necessary to measure both factors, velocity and direction, and reduce to obtain the velocity normal to the cross-section. In small streams or tail-races it is possible to hold the meter rigidly in place; but, when thus held, if the direction of the current makes an angle with the horizontal, that is, strikes the wheel of the cup meter either on the top or bottom of the cups, it will over-register as readily as if running in "eddies" or "boils". The amount of this over-registration depends on the angle which the direction of the current makes with the horizontal, and can only be determined by some check method.

It is not at all clear to the writer why a still-water rating of a current meter should not give an entirely satisfactory result. In all cases where the elements of the current in the open channel are reasonably parallel, the meter is working under almost the same conditions as when drawn through still water. In one case the meter is free and the water moving, and in the other the meter is free and moving, and the water still.

The following test of a direction current meter was made after it had been rated in still water. It was made in a large tidal channel, free from "eddies" or "boils", where the elements of the current were parallel. Each comparison was made by running the meter for 10 min., and in this time running ten large cylindrical can-floats, alternating on each side of the meter. The determined velocities are in feet per second, as follows:

Float.....	1.04	1.29	1.55	1.94	2.56	2.65	2.76	2.80	3.08
Meter....	1.05	1.28	1.55	1.96	2.59	2.64	2.74	2.82	3.11

Mr. Haskell. The greatest uncertainty in the results obtained with cup wheel meters is caused by using such meters from an unstable support, like a boat, particularly if the weather conditions are such as to give the boat any rocking or pitching motion.

The author is deserving of much credit for his work and his paper. Well planned and well executed work cannot help but make for engineering progress.

Mr. Miller. CHARLES H. MILLER, M. AM. SOC. C. E. (by letter).—Although the writer has made scarcely any current meter observations during the past twelve years, and hence has not kept up fully with the progress in such work, he is much pleased at the opportunity to read such a clear presentation of a series of observations, evidently made with much painstaking care; and is gratified to note such a complete verification, stated, however, in a much clearer manner, of a positive difference between a cup meter and a screw meter when running in perturbed water; which difference the writer noted just a short time before leaving the Government service, and has never had opportunity to investigate further.

Although the difference was then cited as being due to instability of support, in other words, the rocking of the skiff, this rocking was done for the purpose, not only of causing conditions actually to be found in the river on account of "boils" and "eddies", but to show the effects that might be expected should the skiff or support be moved otherwise than in a straight and even course, while taking a rating in still water. That the writer was familiar with this source of error a number of years ago may be noted by referring to a letter* written by him, in which, in speaking about the rating of meters in skiffs, it is stated that "a rating should never be taken in a skiff which is being pulled with the oars," and that "the observers should be cautioned about keeping the same positions and not swinging their bodies from one side to the other while the observation is being made." This same passage was noted by the late William Starling, M. Am. Soc. C. E., and referred to in his paper entitled "The Discharge of the Mississippi River."†

Mr. Starling's paper treats in general of the many difficulties attending the taking of discharge observations on the Lower Mississippi River, and explains the reasons for some of the apparently discrepant results published in those days, he having had access, at the time, to the large mass of data gathered by the writer and others then engaged in wrestling with the problem of determining the extent of, or in eliminating, errors. It must be remembered that the possibility of error is very great when measuring the discharge of a river in which a favorable cross-section has a width of approximately ½ mile, with a

* *Engineering News*, January 24th, 1895, p. 58.

† *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 347.

maximum depth of 105 ft. at times, where velocities of 9 ft. per sec. or greater are found, and a total discharge of 1 750 000 cu. ft. per sec. may be obtained. Under such conditions, it is impossible to anchor the boat or equipment, and it must be held on one spot, as nearly as possible, by means of the rudder and the propelling power, counterbalancing the current and wind forces. Careful observations were made to determine how much movement actually took place. In fact, a large portion of the time in the field was devoted to checking the accuracy of the observations. It may be noted that the meters then used were of the cup variety. Some checks, however, were made by a system of double floats. Mr. Miller.

As noting some of the improvements that were being made from time to time, attention is called to the fact that, at a time as recent as 1891, the meter rating was considered as following a straight line, and the proper straight-line equation was calculated by the method of least squares, often, however, not until some time after the field observations had been made. About the date just mentioned it became customary to plot the rating observations and draw the curve conforming as nearly as possible thereto, from which a rating table was taken directly. Other tables were made to cover all necessary computations in the discharge work, and each station observation was calculated while the boat or steamer was moving to the next station, so that the total discharge was known a few minutes after the last velocity observation had been made. For instance, during the morning of May 4th, 1892, at Arkansas City, Ark., the discharge was computed at 1 733 676 sec-ft. as compared with 1 532 875 sec-ft. for the day before. Another observation, made during the afternoon, gave 1 741 526 sec-ft., showing that some marked change had taken place during that time. On the following day it had dropped back to 1 589 172 sec-ft., or to about the general average at that time. Under the old methods, the observations were not computed until some time, often weeks, after the field observations had been made.

As illustrating the fact that every method that could be thought of at the time was used to eliminate errors, it will suffice to say that, in the case of a large and persistent difference in the discharge, as found at two stations 90 miles apart, new meters were purchased, later they were interchanged, frequent ratings were made, and finally the observers exchanged places with each other; all this was done without finding a satisfactory reason for the difference. This was in 1892. During the high water of 1893 these same stations were again occupied, and further efforts were put forth to get accurate results. Mr. Starling, in the paper previously cited, has this to say regarding this work:

"In 1893, the discharges at Arkansas City and Wilson's Point agree fairly well; and when checked by the results of the flanking observations, they present a very coherent mass of data, affording the

Mr. Miller. most trustworthy measurement of the extreme high-water discharge ever obtained on the Mississippi."

Too much stress cannot be placed on the fact that it is just as important to avoid errors in rating meters as in taking the actual discharge observations; and the writer is strongly of the opinion, based on long experience in the field, that by far the larger part of the error is due to the fact that ratings are usually made under physical conditions which are widely variant from those under which the discharge observations are made. For instance, if the observations are to be made from a steamer, then the ratings should be made from the same or a similar steamer, and in dead water, if possible, although they can be made accurately in moving water, provided the movement is smooth, generally uniform, and slow enough to permit the taking of slow velocities in the down-stream direction. Care must be exercised in keeping the meter below the influence of the "drag" occasioned by the steamer hull moving through the water.

Necessarily, the official reports, on account of limited space, as a rule, can contain but a brief explanation of methods and results; and, naturally, these reports incline toward the better showings, failing in general to recognize the fact that the publication of actual failures will often hasten the time of the final solution of the problem.

It may be of interest to know that the first forms of meters were of the wind-mill, screw, or propeller type; that is, the vanes turned, in a plane which was perpendicular to the direction of the current. These were used on South American rivers in 1871.

To eliminate the tail or rudder appears to be a move in the right direction, but a vertical rod could hardly be used with success in gauging a stream as large as the Mississippi, and it would no doubt be possible to work out a design which could be mounted concentrically on the cable which is now used to lower the meter, with its attendant lead weight, to depths of 60 ft. or more, affixing the tail or rudder to the lead weight, and placing the latter so far below the meter that it would not influence its running.

Mr. Hoyt. JOHN C. HOYT, M. AM. SOC. C. E.—Although more than fifty devices for measuring the velocity of water have been patented, the only meters which have had general use are the Price, Haskell, Fteley, and Ellis, or modifications of these.

Each of the various meters has first been devised to meet the requirement of some special condition, and may or may not be applicable to other conditions. Therefore, a current meter should never be used for other conditions than those for which it is designed or has been found to be applicable.

In 1888 the United States Geological Survey began gauging streams of all sizes and in all sections of the country. These streams pre-

sented an infinite variety of combinations of range in depth, width, and velocity, were often remote from railroad or other regular transportation facilities, and were consequently difficult and expensive to gauge. No adequate instruments or methods had been developed for work of this varied nature. Furthermore, elaborate equipment and methods were out of the question on account of the limited funds. It was necessary to devise or adapt a current meter which could be readily carried in the field and operated by one man, either from a bridge, boat, cable and car, or by wading. Mr. Hoyt.

To fulfill these requirements it was essential: (*a*) that the meter should be simple and light in construction, with no delicate parts which easily get out of order; (*b*) that it should be simple in operation, including its preparation for use under any conditions, and its dismantling, cleaning, and boxing after use; (*c*) that it should offer a small area of resistance to the action of the water; (*d*) that it should have a simple and effective device for indicating the number of revolutions of the wheel; and (*e*) that it should be adaptable for use under all conditions.

After experimenting with various types, the engineers of the Survey developed a meter combining certain essential features of the Price acoustic and the large Price electric meter. This is known as the small Price electric meter, and is now in general use in the Survey work. Modifications in its construction have been made from time to time, until now it represents the ideas of many engineers based on results of more than twenty years of experience in stream gauging.

The limitation of the Price meter pointed out in this paper, namely, that it may be affected by cross-currents, has always been recognized by its designers and users. Therefore, special care has been taken to restrict its use to work where the conditions are such that errors from this cause will not affect the final estimates of discharge. These requisite conditions are: (*a*) a fairly smooth bed; (*b*) a measurable and steady velocity of current; and (*c*) a stationary stage during the measurement. The velocity of the current should vary gradually throughout the section, which should show no marked eddies, cross-currents, or boils, and its mean should not be less than 0.5 ft. per sec. at low stages. After visiting hundreds of gauging stations, on streams of varied character, in all sections of the United States, the speaker is convinced that only the exceptional station is not or cannot be located in accordance with these requirements. This statement is made in order to correct any impression which may be drawn from Mr. Groat's paper, that the Geological Survey records, based on measurements with the small Price meter, generally involve large errors. More than two hundred engineers have been engaged in the stream gauging work of the Survey for various periods during the last twenty years, and, as far as known, no valid criticism of the methods used

Mr. Hoyt.

has ever been made which was not fully appreciated by the Survey engineers; and they have made combined effort to correct, as far as possible, each defect in the work as soon as it is discovered. The instruments and methods have been subject to continual change, and old have been supplanted by new whenever the new have been proved the better. Constructive criticism of the work is, and has always been, welcomely received and acted upon, and this opportunity is taken to appeal to engineers who are interested in the use of stream flow records for more constructive criticism and for co-operation in raising this branch of hydraulics to the high plane for which all who are engaged in it are striving.

A few points in Mr. Groat's paper may be referred to briefly:

1.—The paper calls attention to the recognized fact that the differential meter is affected by cross-currents.

2.—It is believed by the speaker that the methods used in carrying on the work from which the conclusions in the paper were drawn, and the magnitude of the results obtained, are both open to question.

3.—Unfortunately, the methods are described only in a general way, and various factors which might affect the results materially are not adequately presented. Furthermore, sufficient data for making any exhaustive study of the results are not given. Therefore, any detailed discussion of the work is impossible.

4.—The large Price meter used in the work is of a type which has not been used for several years by engineers who are most familiar with stream gauging. In the work of the Geological Survey, it was abandoned several years ago, when fifty meters of this style, all practically new, were condemned and replaced by the small Price meter, which was found to give more satisfactory results.

5.—The reader is left in great doubt and uncertainty in regard to the standards used in the comparisons. Though the Haskell meter used may not be seriously affected by cross-currents, there are no experimental data, as far as known to the speaker, which show that this meter gives results any more accurate under the adverse conditions stated than does the Price meter.

6.—The results obtained with an unrated Pitot tube do not appear to be sufficiently conclusive for any final deductions. The Pitot tube itself has had no extended use in open channels, its behavior has not been fully investigated, and, in the limited experiments which have been made with it, many questions have come up which leave an uncertainty as to its action. It has been demonstrated that its rating does not hold near the sides of a canal, and it has also been shown that it will not measure the true velocity when the opening is not in line with the current.

7.—The paper points out a well-known defect in the Price meter,

but gives no remedy, and offers no substitute which can be used under a wide variety of conditions. Mr. Hoyt.

8.—The assumption that the differences shown by the so-called still-water ratings of the meters apply when the meters were used in the canal is questionable. The amount of difference between the Price and Haskell meters, when rated under the conditions mentioned, will depend on the amount of disturbance introduced into the rating; as the Price meter will record, not only the horizontal motion, but also the vertical motion and the cross-currents.

9.—The experience of the Geological Survey shows that accurate ratings of the current meter cannot be made from boats, owing to the errors introduced in connection with obtaining the various factors entering into the rating. Changes in the methods of rating by the United States Geological Survey, the Bureau of Standards, and the Rensselaer Polytechnic Institute have changed the results of ratings about 3 per cent.

10.—The speaker is unable to see why one familiar with current-meter work would have attempted to use the Price current meter under the conditions found at Massena. Engineers of the Geological Survey have frequently refused to co-operate in work where similar conditions existed.

11.—The statistical synthesis method, as understood by the speaker, does not take into account the relative distribution of velocities and areas in various parts of the cross-section. The mean velocity in a cross-section, as defined by the Geological Survey, is obtained by dividing the discharge by the area, which will not give the same results as taking the mean of the velocities at various points in the cross-section. For example, a certain stream may have 50% of its flow in 10% of its cross-section. Furthermore, this method of computation has no apparent advantage over that ordinarily used, and has the disadvantage of being much more complicated, even if the results are of equal value.

12.—The question of the distribution of velocity in the vertical has been investigated most thoroughly by the engineers of the Geological Survey, who have found the mid-depth method to be the least desirable. The mean of the velocities at two-tenths and eight-tenths of the depth gives (without a coefficient) the velocity in the vertical, under all conditions of natural flow; and, next in value, the six-tenths depth method has been found to be the most satisfactory.

The only method yet devised for rating a current meter is by drawing it through still water. A close study indicates that no material error is introduced in the results obtained with this instrument on account of the method of rating. Large numbers of comparative measurements have been made with the Price current meter, calibrated weirs,

Mr. Hoyt. and measuring tanks, all of which show that the meter, when properly used, will give results accurate within 1 or 2 per cent.

It has also been found that measurements at different points on a stream, where the conditions of measurement are entirely different, give results which agree closely. Many measurements have been made on two tributary streams and then on the main stream below, and the results check within the limits of accuracy necessary in work of this type.

Mr. Staniford. C. W. Staniford, M. Am. Soc. C. E.—The speaker would like to ask Mr. Groat or Mr. Hoyt whether any of the current meters mentioned—the Haskell, the Price, the Ellis, or perhaps the admirable German instruments—after being rated in fresh water (still water, of course), have been used in salt water; and what percentage of accuracy can be obtained with such meters in salt water?

If such instruments have been rectified against corrosion, and against short-circuiting, can they be relied on when used in salt water?

Mr. Groat. B. F. Groat, Assoc. M. Am. Soc. C. E. (by letter).—If one will examine the papers and discussions concerning current meters published since the original study was made by Frederic P. Stearns, Past-President, Am. Soc. C. E., in 1879,* it will be found that there is much uncertainty among engineers concerning the reliability of these instruments, with which, up to the present time, there does not seem to have been any generally satisfactory and generally adopted practice. These facts are in sharp contrast to those connected with the use of other engineering instruments, such as the transit and level. No method can be deemed generally satisfactory until it meets the general approval of practitioners. So long as there is lack of agreement between various classes of current-meter observers, not only as to the general method of procedure, but also as to the reliability of various types of instruments, there can be no such thing as a satisfactory current-meter practice.

When it becomes generally realized that current meters obey their still-water ratings only when the conditions of flow are ideal, and that cup meters are affected in one sense and screw meters in the contrary sense, it will be apparent that there is a real reason for the diversity of opinion concerning the reliability of current-meter work.

The writer, therefore, was inspired by the hope that this paper might reopen the discussion and possibly put in motion an effort to improve the current meter.

Appended to the paper there is a brief description of a new system of taking discharge observations and a new method of constructing the discharge curve. This system and method have been applied most

* *Transactions*, Am. Soc. C. E., Vol. XII, 1883, p. 301.

successfully in a number of cases, and are worthy of the attention of those contemplating a series of gaugings at one or more stations. Mr. Groat.

The method of synthesizing a discharge curve is not an invention of the writer which he is trying to introduce, but is a well-established method used by physicists and practicing engineers in determining various kinds of relations between observed magnitudes. It consists simply in separating a given investigation into its elementary parts and drawing a curve for each, the final relations being derived from the elementary curves or relations. The fact that the sum of the velocities at a given set of meter points at a gauging station forms such an elementary relation, when plotted as a function of the gauge height, leads to the term "statistical."

The main advantage of the statistical method is that the entire series of gaugings, extending over a long series of years, may be connected into a consistent whole. The peculiar characteristics of a station may be studied from the elementary curves. If any change or break occurs in one or more of the elementary curves, a corresponding change or break has taken place in one or more of the physical characteristics or hidden conditions of the station. This is especially true of the sum of the velocities at a given set of meter points, because the sum forms a fixed relation between velocity and stage. The sum is chosen because, in taking a number of velocities, the accidental variations and errors in the velocities eliminate themselves. That is why, even at turbulent stations, a smooth curve can be drawn through the plotted values of the sum, with errors well within 1 or 2%, as extremes. It is not likely that changes in the hidden conditions at a station would be detected by any other method. They would be set down simply as unaccountable variations.

Another very important advantage of the statistical method is that it affords at once the most independent and most easily secured check on the velocity readings. Indeed, the sum of the velocities must plot on a definite curve, and this enables one to check the meter work at any newly observed stage with that at any other stage within 10 min. after all the velocities for the former have been recorded.

The writer is glad to know that his Conclusion 1, concerning cup meters, is generally recognized. Under certain nearly ideal conditions, such meters will give good results, but, the moment the conditions are changed in the slightest degree, the still-water rating becomes more or less inapplicable, with a resulting constant or systematic error.

Nevertheless, it would be perfectly safe to use a cup meter in turbulent water if, where used, it were rated with a Pitot tube. The fact that the meter runs with fidelity under such circumstances is another of the conclusions drawn in the paper, and one should not lose sight of this good quality. It may also be remarked that this is a

Mr. Groat. fact which does not seem to have been generally recognized. Cup meters will undoubtedly have their uses, but improvements in the means for obtaining their ratings—not still-water ratings, but actual ratings at the time and place of use—are imperative. It will do to talk about refined ratings when probable errors are reduced to values of less than from 3 to 5 per cent.

After reading the discussions one can scarcely shake off the idea that the niceties demanded for the conditions under which cup meters must be rated are altogether out of proportion to the degree of accuracy attainable with these meters in stream gauging. If a cup meter cannot be rated accurately from a boat when operated carefully, owing to changes in rating due to the slight irregularities of motion, what must be the effect on such a meter when subjected to the far greater irregularity of the motion of the water at a fairly good gauging station, to say nothing of the errors to be expected at a really turbulent station.

The writer believes that Mr. Miller made an important discovery in current-meter engineering when he noticed that cups over-register considerably and screws under-register slightly in agitated water. In due time this fact will impress itself on engineers generally, and lead to a meter which obeys its still-water rating more closely than any hitherto made.

The Haskell meter is so nearly correct under a wide range of conditions that it would seem to require only slight modification, as suggested in the paper, to render it almost independent of conditions. It would be a delightful task, for some one who has the time and means, to develop such a meter. Meantime, it cannot be too strongly urged on hydraulic engineers that a screw current meter is the most reliable kind for measuring the velocities of flowing water when using the still-water rating.

It may be well to define the terms "cup" and "screw," as the writer does not wish to apply the general propositions stated to every kind of instrument which may chance to be called a cup meter or a screw meter: A cup meter is one in which the rotating element operates on the principle of the Robinson cup anemometer, though the shape of the cups need not be limited to that of the hemisphere. A screw is a rotating element the surface of which would be generated by a straight line perpendicular to, rotating uniformly about, and moving uniformly along, the axis of rotation.

One would also hesitate to apply the propositions to meters having large or badly designed frames which would be likely to interfere with the action of the water on the runner. Although the frame of the cup meter is at the rear of the runner, it has decided and different effects on the rating of the instrument when turned somewhat to the right and then an equal angle to the left of the line of motion.

Mr. Groat.

The writer is more than pleased to have Mr. Price's open-minded discussion. The cup meter is very ingenious, and the means which Mr. Price has adopted to protect the bearings and secure uniformity of friction have probably been the cause of the continuance of this meter in the Government service.

Mr. Price may be assured that the meter was suspended in precisely the same manner during the turbine tests as in the ratings. This applies also to the stream tests. No weights were used in any case, and the meter wheel was about 7 ft. in advance of the stem of the rating boat. The reaction of the boat on the screw must have been slight indeed, as the ratings from a car at the Rensselaer Polytechnic Institute do not differ materially from those by the boat.

Fig. 1 shows the rating boat, and below it the rod used to support the meter. The hooks on this rod may be adjusted so that any desired series of depths may be secured. From the staging on which the observer stood a $\frac{5}{8}$-in. rod extended across the tail-race at a fixed elevation. The meter was attached to one end of a $\frac{3}{4}$-in. rod, about 2 ft. long, the other end of which was screwed into the supporting rod. There were seven hooks on the supporting rod during the turbine tests, any one of which could be hooked over the $\frac{5}{8}$-in. rod on the staging, thus immersing the meter to the corresponding depth. Six rope clips were attached to the $\frac{5}{8}$-in. rod at the verticals in which the meters were to be operated. The rope clips served as stops at which to hold the meter rod, when taking observations in the verticals.

Fig. 2 is a rough sketch of the Pitot tube and supporting rod. The Pitot tube (to the right) is a copy of "Tube *N*," as described in Mr. White's monograph, referred to in the paper. The supporting rod (to the left) was designed to be operated from the staging in the tail-race, in the same manner as the supporting rod for the meters. On Fig. 2 is shown one of the hooks to engage the $\frac{5}{8}$-in. rod on the staging, and also the eyes on both the Pitot tube and meter supporting rods to which a guy wire was attached near their lower ends, thus holding the rods and preventing bending.

Mr. Price states that the meter of the propeller type does not free itself from such débris as dead grass and leaves. This is too broad a statement, for there is at least one meter of that type which does free itself, and that is the Haskell. This meter has been used by the writer many times in streams carrying much dead grass, leaves, twigs, sand, and mud. On no occasion did it catch the slightest particle or give any evidence of being affected thereby.

This is due to the effective shape of the runner, which Dean Haskell has undoubtedly designed expressly for the purpose. The up-stream end of the wheel is brought down to a moderately pointed or conical nose which sheds all débris carried by the water.

The question of friction in a current meter is very interesting. Throwing out the case of low velocities, the necessity seems to be,

Mr. Groat. not so much for a small amount of friction as for constancy of amount. If the resisting torque is large but constant, a rating should be fixed just as exactly as in the case where the friction is small and constant. Notice, too, that there is a resisting torque caused by the hydraulic skin friction on a screw, which can be made as large as desired by lengthening the vanes along the axis. In this respect, the screw is just as much a differential meter as the cup meter.

The retarding skin friction has a much longer lever arm than the bearing friction, and has a much larger resisting torque, so that the bearing friction of a screw meter offers a resistance of secondary importance. Further, the resisting skin friction is a fixed function of the velocity, varying approximately as the square thereof. Hence it is clear that the principal portion of the resisting torque is fixed for each velocity and cannot be changed. It follows that, if the meter is kept in only fair condition, the rating is not affected seriously by moderate changes in bearing friction. The truth of this was tested in the experiments at Massena by taking part of a rating while the bearing was dirty and stiff, then completing the rating after the meter had been cleaned and oiled. Let the reader select from ratings *E* and *F* of the Haskell meter, Plate XXIII, the points taken before cleaning and oiling the meter. The Haskell meter is not sensitive to moderate changes of bearing friction.

Mr. Price thinks the ratings (Plate XXIII) of the Haskell meter are inaccurate. If any one will take the pains to examine the percentage errors of the various ratings and the wide range of conditions under which these ratings were made, he will feel compelled to admit that the performance of this meter was very consistent throughout. Ratings of this kind are of more practical value than those obtained under restricted conditions, because the former afford an indication of the degree of accuracy to be expected of the meter when in actual service, while the latter do not.

No doubt the rating line could be adjusted to the observations somewhat better. This rating line was located from a few of the earlier ratings, the later ones being plotted on the diagram subsequently. The change, if made, would be trifling, excepting for low velocities, which were not used.

The idea must not be entertained that the conditions in the Massena tail-races were bad. On the contrary, they were much better than those usually encountered in stream gauging. The flow of the water was parallel to the side-walls, and the meters might as well have been fixed rigidly to the rods, as far as concerned any tendency to swing. The errors of the meters were caused by small eddies and boils such as would certainly have to be dealt with at most gauging stations. The conditions of flow were carefully studied at each of the 84 meter points by using an inverted weather vane. An instrument

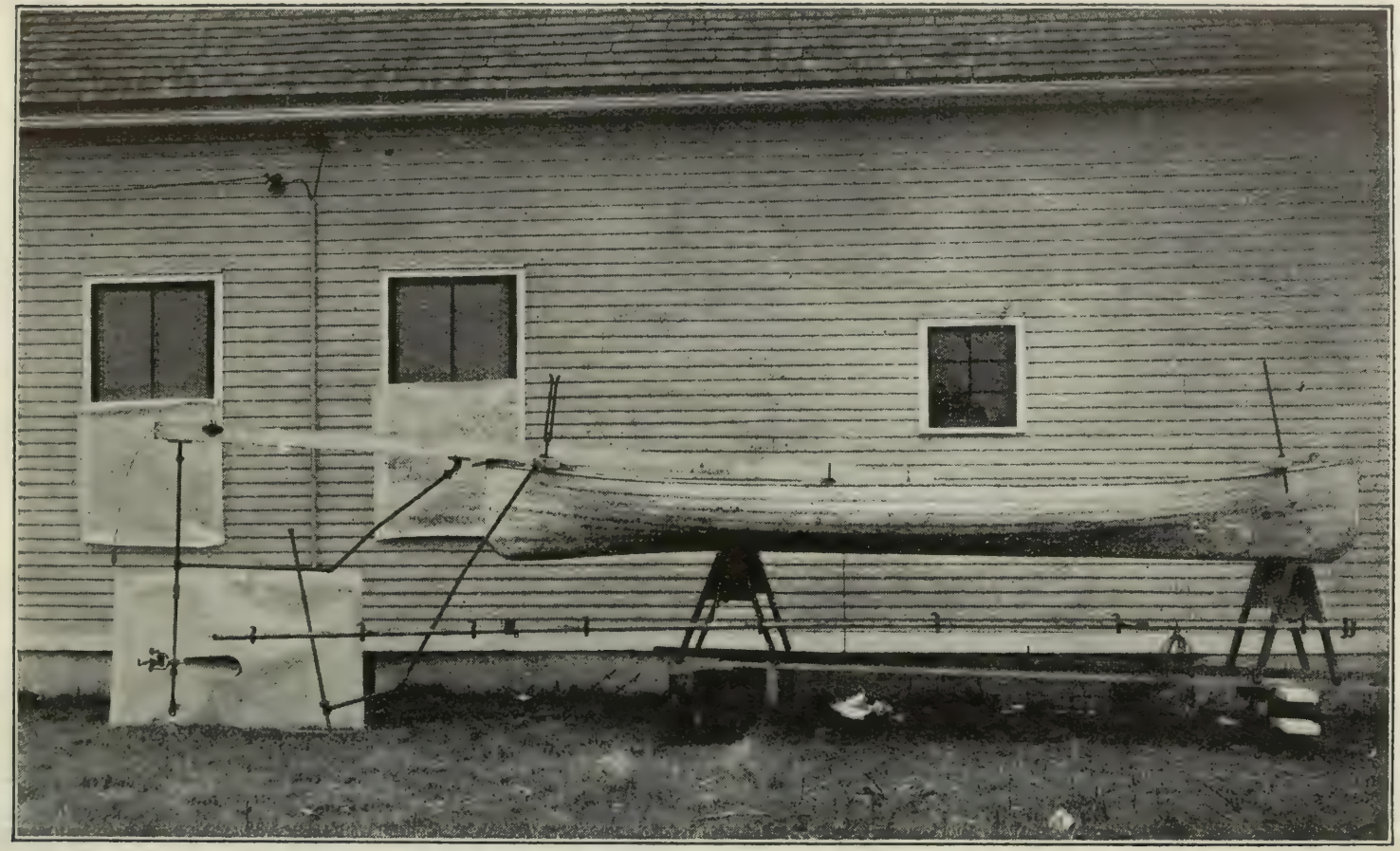


FIG. 1.—RATING BOAT, WITH ROD USED TO SUPPORT CURRENT METER.

was invented and used to ascertain the quantity of air in the water at each meter point. This quantity was found to be negligible. Mr. Groat.

Mr. Price states that a gauging station which has an up-stream current should never be used, and one which has divergent currents cannot give accurate results with any method of measurement. It

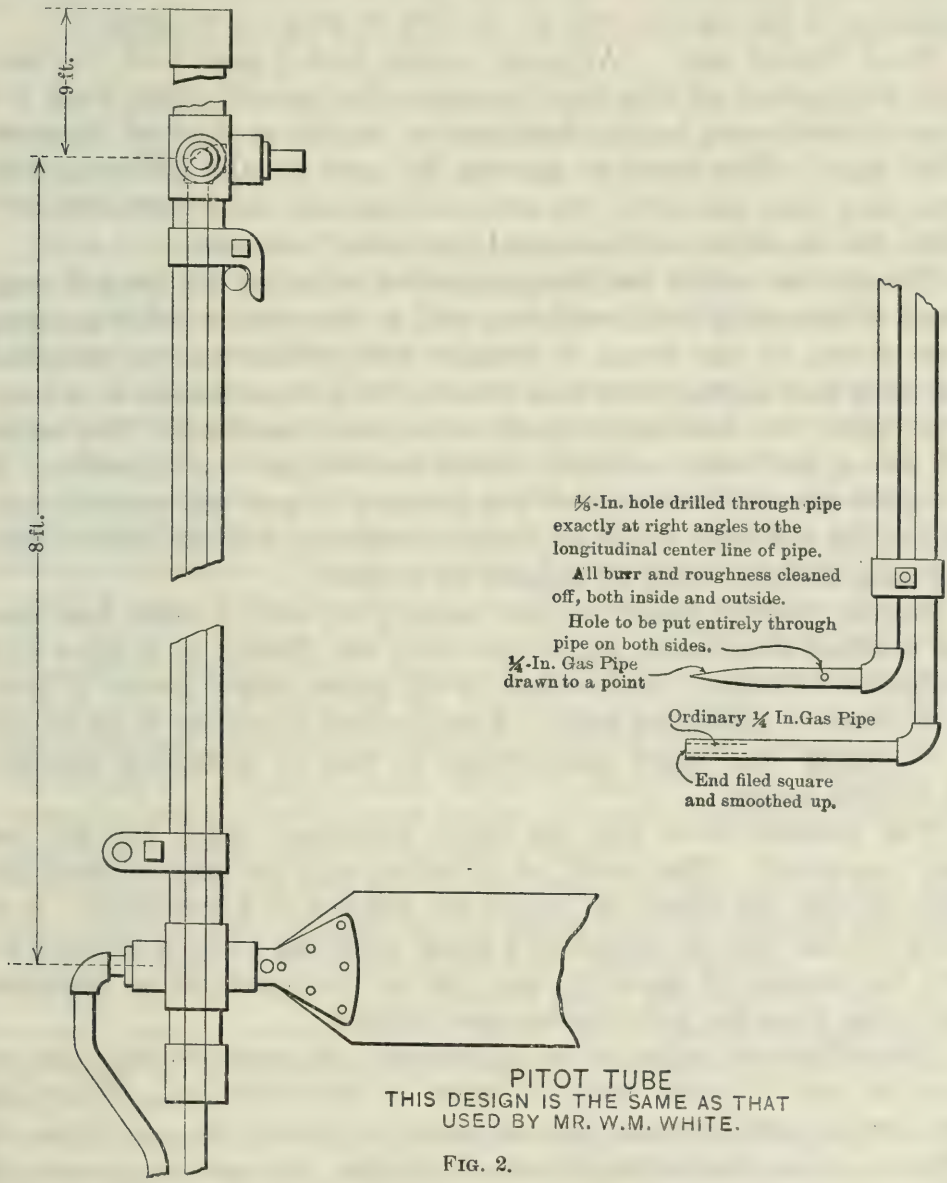

9-ft.
8-ft.
⅛-In. hole drilled through pipe exactly at right angles to the longitudinal center line of pipe.
All burr and roughness cleaned off, both inside and outside.
Hole to be put entirely through pipe on both sides.
¼-In. Gas Pipe drawn to a point
Ordinary ¼ In. Gas Pipe
End filed square and smoothed up.
PITOT TUBE
THIS DESIGN IS THE SAME AS THAT USED BY MR. W.M. WHITE.

FIG. 2.

may be that gauging stations with negative velocities should not be used, but do our current meters give any indication which would lead one to discard such a station when encountered? If the meter is 4 or 5 ft. under water, it cannot be seen; but it goes on recording the velocity just the same, even if the current is directed up stream,

Mr. Groat. and the unsuspecting observer records a considerable positive velocity at the point, whereas it should have been recorded as negative.

As to the matter of divergent currents, the writer believes most fully that the problem may now be solved by using a screw meter held rigidly at right angles to the direction of flow. When the screw is modified, as suggested in the paper, it will increase still further the reliability of the results, even at the best of gauging sections.

Dean Haskell says: "Although a great deal of good work has been done with meters of this type [meaning cup meters], they have not shown themselves to be anywhere near as reliable as those of the screw wheel type." This seems to describe the case exactly. Where conditions have been favorable, the errors of the cup meter have not been large; but the errors are there, and they are of one sign.

Though the writer has been compelled to adopt the method suggested of observing the direction as well as the velocity, when gauging large rivers, he has found it fraught with difficulties, the principal one being that he has never been able to find a cross-section on a large river where the direction at each meter point was fixed. The meter will swing, and this completely masks the average true direction. If the meter was held rigidly, and was designed to give the resolved component, the difficulty would be obviated instantly, and the observations and computations would be reduced in number.

As the Haskell meter as now made gives only a trifle less than the resolved component, the writer takes the liberty to suggest that a slight modification, as described in the paper, might render it absolutely exact in perturbed water. A mechanical advantage to be gained by "cupping" the wheel vanes would be that of additional strength for a given weight.

The Haskell meter has one other advantage which has not yet been mentioned: The screw, or propeller, may be made exceedingly light, so that the effects of inertia are reduced to a minimum. It is quite possible that the inertia of a heavy revolving part, in conjunction with the reaction of the water, may play an important rôle in changing the rating when the velocities become variable.

The still-water rating is not satisfactory at present because meters must be used in water which is more or less perturbed. Otherwise, the current meter becomes an instrument of limited use, and demands attention to a complicated system of details. As soon as a meter is produced which will resolve the momentary velocities at a meter point, the still-water rating will apply, even in perturbed water. Dean Haskell is to be congratulated on the fact that his meter almost does this very thing, so that probably only slight modification will be necessary to secure the desired result.

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The float and meter comparisons are interesting. Applying the statistical method of summing the velocities in each case, the resulting comparison follows:

Sum of velocities by float	19.67
Sum of velocities by meter	19.74
Difference	0.07

Mr. Miller's discussion is very interesting, as it recounts work of unusual magnitude on the Mississippi. The writer has lived at various points on that river all his life, and remembers much of the Government work of various kinds there.

It is to be regretted that such a keen observer as Mr. Miller should have left the field of current-meter engineering. However, a useful fact once established is bound to live, and thus Mr. Miller is still potentially active in this branch of science.

Mr. Hoyt seems to fear that an impression may be drawn from the paper that the Geological Survey records, based on measurements with the small Price meter, generally involve large errors. This fear is hardly justifiable, as the writer states very definitely, "it seems certain that the correction for a cup meter when run at a good meter station on an open river is not large." On the other hand, it would certainly be on the side of safety to determine the degree of error in every important series of gaugings. One of the best ways to uncover an error of this kind is to run a cup meter and a screw meter in the section simultaneously. If the meters agree, there is no error. If they disagree materially, it will be found that the velocities by the cup meter are excessive, and those by the screw are deficient, the numerical value of such errors by the cup meter being relatively larger than those corresponding by the screw meter.

If any one wishes adminicular evidence as to the truth of this statement, beyond that afforded by the paper, he may refer to the paper, "Current Meter and Weir Discharge Measurements,"* by E. C. Murphy, M. Am. Soc. C. E. In this paper Mr. Murphy has, in the clearest manner, plotted the results of a number of comparisons of discharges by a weir with simultaneous discharges by cup and screw current meters.

In the case of the cup meters, practically all the discharges exceed the corresponding discharges by weir. This is indicated by the fact that practically all the points for the cup meters plot below the axis of abscissas in Fig. 2 of that paper.

In the case of the screw meter, the discharges are generally deficient, as the points generally plot above the axis of abscissas. Moreover, this

* *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 376.

Mr. Groat. is the more accurate meter, because the points average much closer to the axis, being in fact within 0.07% of it.

Mr. Hoyt concludes his discussion with several numbered paragraphs. They may be taken up in the same order:

1.—The paper does much more than call attention to the recognized fact that the cup meter is affected by cross-currents. It calls attention to the fact that the effect is productive of errors all on the side of excessive velocity, and that the screw meter is affected in the contrary sense, but to a relatively smaller extent. Though this was discovered by Mr. Miller years ago, it does not seem to have been recognized by anybody else since that time; but it is undoubtedly of the greatest importance to those who wish to improve the present methods of stream gauging.

Before carrying out any experiments at Massena, the writer was of the opinion that screw meters, like cup meters, would always show excessive discharges. This, he thought, would be the case because of the swinging of the meter and the consequent measurement of the thread of a sinuous, rather than of a straight, current. This, however, is incorrect. To prove it, attach a screw meter to the rating boat or rating car. Draw the meter over its course in the usual manner and count the revolutions. Then repeat the process, having all conditions the same as before, except that the meter is oscillated transversely to the direction of motion during its transit of the course. If the rating boat is used, the oscillation may be effected by rocking the boat. In the case of the car, some mechanical device may be used.

Theoretically, the number of revolutions in the latter case should exceed those in the former case. Practically, the reverse is the case. The side-wash of the current has somewhat more than annulled the effect of increasing the length of the course.

2 and 3.—It is true that the methods used by the writer in making his discharge measurements were described only in a general way. Otherwise, the paper would have become so voluminous that the main facts would have been obscured. It must be remembered that the conclusions of the paper are not derived from a study of current meters *per se,* but are inductions from observations on the behavior of meters while testing turbines.

4.—The large Price meter was adopted as one type of meter to be used in the Massena tests, because the same type had been used by other engineers in the same tail-races several years before, and a comparison was desired. The results of the turbine tests would have been the same had the small Price, or any other cup meter been used instead of the large Price meter, because means were taken to correct the errors due to deviations from the still-water ratings.

5.—The standard adopted by the writer when comparing the cup and screw meters was the true velocity of the water. This appears when

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discussing Table 5, where it is shown that the cups over-register, on the average, by 5.6%, and the screw under-registers, on the average, by 0.93 per cent.

When the Pitot tube and Haskell meter were compared, the Pitot tube was supposed to indicate, on the average, the true velocity. The result of this comparison was that the Haskell meter under-registered by about 0.6 per cent.

Thus there were two different methods of determining the discharge: (*a*) by cup and screw meters corrected through the medium of the boat-rocking tests, and (*b*) by Pitot tube and screw meter comparisons by which corrections were determined to be applied to the metered discharges.

It is worthy of note that these two methods of correcting the discharge, when applied to any given turbine test, give rise to a discrepancy of scarcely 0.3 of 1 per cent.

It is not claimed that rocking the boat is the best means for ascertaining the relative errors of cup and screw meters. It is claimed that the nature of the case is such that extreme accuracy in this respect is not necessary. This is because the errors to be determined are not relatively large.

If it is feared that this paper is too daring in its conclusions to be credible, it may be well to search for independent verification. Such material may be found in Mr. Murphy's extremely valuable paper, "Current Meter and Weir Discharge Measurements," previously quoted.

Though Mr. Murphy gives an interpretation of the results of his experiments, yet he expressly states that he has "avoided in this paper a detailed comparison of results by different meters." Hence it will be permissible to analyze the results for the purpose of such a comparison.

The best method of eliminating accidental errors in comparing such a series of observations is the "statistical" method. In the case at hand, this method is particularly facile because comparable results may be obtained by the simple operation of summation.

In order the better to understand the conditions under which the tests by Mr. Murphy were conducted, it may be stated that they were made in the canal of the Hydraulic Laboratory at Cornell University. The discharge in each test was measured by weir and by two current meters. The two current meters were observed simultaneously in the same discharge section, one starting on the south side and progressing north while the other started on the north side progressing south.

There were twenty-five experiments, each made in the manner described. Ten of these were comparisons between the weir, Haskell meter No. 3, and small Price meter No. 363. Nine were comparisons between the weir, small Price meter No. 351, and small Price

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meter No. 363. Six were comparisons between the weir, Haskell meter No. 3, and small Price meter No. 351. Moreover, from these experiments may be selected sixteen in which the Haskell meter is compared with the weir, fifteen for Meter No. 351, and nineteen for Meter No. 363.

There are, therefore, six sets of experiments which may be compared statistically. In each case the aggregate discharge by the weir may be compared with the corresponding aggregate, or aggregates, by the meter, or meters, as the case may be. Table 9 gives the comparisons in tabular form.

TABLE 9.—STATISTICAL SUMMARY OF TWENTY-FIVE EXPERIMENTS BY E. C. MURPHY, M. AM. SOC. C. E., IN WHICH THE DISCHARGE OF A CANAL IS MEASURED SIMULTANEOUSLY BY A WEIR AND TWO CURRENT METERS, THE LATTER OPERATED IN THE SAME DISCHARGE SECTION.

Compiled from Table 1, *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 373.

No. of experiments.	Sum of all the discharges in all experiments, as shown by weir, in cubic feet per second.	Sums of the Discharges in All the Experiments, as Shown by the Various Meters.								
		Screw Type.			Cup Type.					
		Haskell Meter.			Price Meter No. 351.			Price Meter No. 363.		
		Sum, in cubic feet per second.	Error, in cubic feet per second.	Error. Percentage.	Sum, in cubic feet per second.	Error, in cubic feet per second.	Error. Percentage.	Sum, in cubic feet per second.	Error, in cubic feet per second.	Error. Percentage.
16	3 474.78	3 473.99	— 0.79	— 0.02	.....	.....	....	.....	.....	....
15	3 373.67	.....	....	....	3 439.03	+ 65.36	+ 1.94	.....	.....	....
19	4 186.17	.....	....	....	.....	.....	....	4 238.23	+ 52.06	+ 1.24
10	2 143.64	2 147.39	+ 3.75	+ 0.18	.....	.....	....	2 164.22	+ 20.58	+ 0.96
6	1 331.14	1 326.60	— 4.54	— 0.34	1 368.26	+ 37.12	+ 2.79	.....	.....	....
9	2 042.53	.....	....	....	2 070.77	+ 28.24	+ 1.39	2 074.01	+ 31.48	+ 1.54

The function of such a summary of aggregates is to eliminate in large measure the unavoidable accidental errors arising during the observations. Thus the remaining errors are each composed of two parts, namely, a large part, which is the net average constant error of the observations, and a small part, which is the net residual of the unequated accidental errors. From the approximate uniformity of the errors of the table, it may be concluded that accidental errors have been practically eliminated, and the computed errors for the sets of observations of greatest weight (the first three sets in the table) are the constant errors due to the meters, or due to deviations from their still-water ratings. Thus the constant error of the Haskell meter is — 0.02%, that of the small Price No. 351 is + 1.94%, and that

of the small Price No. 363 is + 1.24 per cent. It may be inferred from the plottings by Mr. Murphy, on page 376 of his paper, that the average constant error of No. 351 should exceed that of No. 363. The reason for this is that the discharge by either of the meters shows an increasing excess over the discharge by the weir with decreasing velocities, and there are relatively more observations of low velocity by Meter No. 351 than by No. 363. Had the reverse been the case, it might have been expected that the constant error of No. 363 would exceed that of No. 351. Mr. Groat.

It will be seen that the error of the Haskell meter for the ten comparisons with weir and small Price meter No. 363, is + 0.18 per cent. This error, though insignificant, would have been negative had it not been for three experiments, Nos. 1-2, 3-4, and 5-6, in which this meter, for some unaccountable reason, gave discharges 4 or 5% greater than the weir. These three observations are plainly discordant, as may be seen by referring to the plottings for the Haskell meter in Fig. 2 of Mr. Murphy's paper. Had the meter run in these three experiments according to the same law as in all the other experiments, the errors for these three experiments would have been about nil. Thus the excess in discharge by the Haskell meter in the three experiments in question is about 25 cu. ft. per sec., as may be seen by consulting Table No. 1 of Mr. Murphy's paper. The aggregate for the weir from the writer's comparison, Table 9, is 3 474.78, and that for the meter would become 3 473.99 — 25.0 = 3 448.99. Thus the deficiency in the aggregate for the Haskell meter would be 3 474.78 — 3 448.99 = 25.79. In other words, it is more than probable that the discharges by the Haskell meter are deficient by the slight percentage of 25.8 ÷ 3 475 = 0.75. However, it will not do to correct or reject observations simply because they are discordant.

It follows from the results shown in Table 9 that the cup meters in all cases gave excessive discharges and the Haskell meter gave relatively much smaller errors, the discharges being slightly deficient on the average. Therefore, the following general conclusions may be drawn from the results contained in Mr. Murphy's valuable paper:

(*a*) When a cup meter is run in moderately perturbed* water, it will register a larger number of revolutions per second than a perfect still-water rating would indicate.

(*b*) When a screw meter is run in moderately perturbed water, it will register a smaller number of revolutions per second than a perfect still-water rating would indicate.

(*c*) In the foregoing sense, a cup meter is affected relatively to a much greater extent than a screw meter.

* At the bottom of page 374 of Mr. Murphy's paper it is stated that there was a slight agitation of the water during the experiments.

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(*d*) Either type of meter, when run in moderately perturbed water, will give uniform records in equal times, provided these times are sufficiently long, the flow of the water itself being subject to an established regimen.

(*e*) If both types of meter are used simultaneously in moderately perturbed water, the disparity between the discrepant velocities thus determined by the still-water rating may be taken as a basis for correcting the discrepant velocities.

(*f*) If the discharge by the weir be considered correct, the average error of the Haskell meter in Mr. Murphy's experiments was 0.02% on the side of deficiency, with a probability of its being slightly larger, and the average error of the cup meters was in the neighborhood of 1½% on the side of excessive discharge.

(*g*) It would seem to follow that current-meter observations based on still-water ratings, without further correction, should be made with great caution. On the other hand, it seems certain that the correction for a cup meter when run at a good meter station on an open river is not large, and the corresponding correction for a screw meter may be negligible.

6.—It cannot be said that the Pitot tube used at Massena was an unrated instrument. It is an exact copy of one which was investigated as "Tube *N*" by Mr. White in the paper already referred to. It has a coefficient equal to unity.

In speaking of the accuracy of Pitot tubes, two distinct reactions must be kept in mind. One is the impulse of the moving water on the dynamic orifice, the other is the reaction of the static pressure and the moving water on the static orifice. If the static orifice is properly designed, the motion of the water, or the dynamic reaction, will have no effect on it; but the motion of the water will exert either suction or a positive pressure on an improperly designed static opening. It is this latter difficulty which has caused most of the uncertainty as to Pitot tubes.

Mr. White's experiments showed, in the most convincing manner, that the coefficients of all dynamic orifices of circular section, be they cylindrical, diverging, or converging, are exactly equal to unity. This was accomplished by towing a boat through the water with a number of different types of circular dynamic orifices attached somewhat in advance of the stem. The water stood at exactly the same level in all the risers, and the total head raised was equal to the sum of the velocity head and static head in each case.

7.—Although the paper does not essay to offer at once a complete remedy for the erratic behavior of current meters in perturbed water, it does point out the interesting fact that a close estimate of the true velocity may be made by using a screw meter of good design. If still

greater accuracy is desired, both cup and screw meters may be used, the records being corrected by the observed relative deviations produced in some such manner as the rocking tests described briefly in the paper; or, even better still, the screw meter may be rated by a properly constructed and properly manipulated Pitot tube, under the exact conditions and at the exact point, or points, where the meter is to be used. Mr. Groat.

8.—The writer has oscillated a number of meters of different types, both longitudinally (up and down stream) and transversely (athwart stream and vertically). In all cases the cups were affected by increments several times as large as the decrements of the screws. This was for violent, as well as moderate, vibrations. It is true that the ratio is a variable, values ranging from 3 to 6 having been noted; but, when the error between the cup and screw is not more than 7%, the application of the extreme ratios leads to corrections for the screw meter of only $1\frac{3}{4}$ and 1 per cent. Thus, an error of 100% in the true value of the ratio makes a difference in the final discharge of only $\frac{3}{4}$% for the case cited.

This method, of course, is only a practical way of getting an approximate result, but the approximation is shown to be close.

9.—A current meter which cannot be rated with sufficient accuracy from a boat will show even greater discrepancies when run at the average gauging station. The United States Lake Survey has done a great work on the Niagara and St. Lawrence Rivers, and its ratings were frequently made from a boat. The writer knows that screw meters were used extensively on this important work, but does not know whether cup meters were also used. If the boat ratings had been unsatisfactory, it is certain that that fact would have led to a condemnation of such ratings. Possibly, if cup meters had been used, the boat ratings would have proven unsatisfactory. Of course, a rating made under laboratory conditions will plot almost exactly on a smooth curve; but a rating of a screw meter of good design will be nearly as good, if made from a boat.

It is presumed by the writer that the changes in the method of rating referred to by Mr. Hoyt were such as might be made by placing the weight too close to the wheel, or by running it near the sides, or bottom, of the tank. These are abnormal ratings, of a character which does not affect the question at hand.

10.—It has already been explained that the large Price meter was selected for the tests at Massena for the sake of comparison with the results of a former test in the same tail-races with a meter of this particular type. The writer is of the opinion that the results would have been substantially the same, even if the small Price meter had been substituted for the larger pattern. It must be emphasized that the conditions of discharge for much the greater portion of the

Mr. Grout. gauging section at Massena were as good as those most frequently encountered at a good gauging station. The reliability of these tests cannot be questioned on this account.

The cup meter ran with great fidelity at any given point in the race. Its rating there, relatively to that of the Haskell meter or Pitot tube, was perfectly definite, a fact which does not seem to have been known heretofore. The trouble was not with the cup meter, but with the application of the still-water rating to the case. A raceway of uniform section can certainly be gauged within 1% of the true discharge. This statement is made because any given condition of discharge can be exactly reproduced at the observer's pleasure. This is one of the facts demonstrated at Massena.

11.—The distribution factor used in the statistical method is essentially a form factor. Its value depends on the shape of the solid constructed out of the components of velocity which are normal to the cross-section when these components are erected at their points of application. To every shape a particular value of the distribution factor attaches, but it is not true, conversely, that to every value of the distribution factor a particular shape must attach. It is only necessary that shapes be equivalent as to discharge in order that two or more shapes may have the same value for their distribution factors. Hence it is not at all improbable that such changes of distribution actually take place at gauging sections without changing the discharge curve or rating table.

The mean velocity in the cross-section is the quotient obtained by dividing the total discharge by the area of the cross-section. The statistical method relies on this definition. In the turbine tests at Massena, the value of the distribution factor was ascertained by drawing the familiar contours of equal velocity over the cross-section of the race. The volume under these contours was computed, and, when expressed in proper units, was numerically equal to the discharge. This discharge was then divided by the area in order to ascertain the mean velocity.

By definition, the distribution factor is the ratio of the above determined mean velocity to the sum of the velocities recorded at the 42 meter points in the race. By actually computing this ratio, in the foregoing manner, for several of the turbine tests where the conditions were substantially the same, it was found that the value of the ratio—that is, of the distribution factor—was constant for given conditions. Consequently, it follows that the mean velocity in the tail-race may be ascertained simply by multiplying the sum of the observed velocities by the proper value of the distribution factor.

Though a somewhat different procedure was adopted in the Southern work, it was clearly proven that the distribution factor is a definite function of the gauge height, or stage of the river, which,

when once reduced to a curve, may thereafter be used as the quickest possible means of ascertaining the mean velocity in the gauging section. This is accomplished simply by multiplying the sum of the velocities at a given set of meter points by the corresponding value of the distribution factor, as read from its locus. Mr. Groat.

In short, it has been demonstrated by these experiments that, in current-meter work, and discharge gauging in general, one may deal with aggregates of velocity area, and discharge, instead of individual values, with a gain in accuracy, time, labor, and satisfaction, to be realized in no other way.

Take, for example, the case of an observer who has just hurried to a gauging station to measure a discharge at a particular stage of the stream. He drops his meter at two or three dozen permanently established meter points (supposing the river is, say, 250 ft. wide), adds up all his velocities, compares the sum with his statistical velocity curve, finds the sum correct, and then multiplies it by a factor taken from a curve, the product being the discharge. The whole work may be done within an hour or two. Moreover, the observer knows his work is correct, because his velocity observations are checked.

12.—It is very certain that the Geological Survey has done excellent work in studying velocity curves and in formulating therefrom a few simple laws which govern the discharge of rivers. Much of this work is due to Mr. Hoyt with whose name is linked some of the most valuable hydraulic investigations.

For the most part, the writer's studies have been connected with shallow turbulent rivers, as he is frequently compelled to examine streams of this class with regard to the character of their flow. It may be of interest to know that he has arrived at two conclusions connected therewith. They are:

(*a*) A shallow turbulent stream may be gauged satisfactorily with a screw meter, or, better still, with a cup and a screw meter simultaneously.

(*b*) The proper point in a shallow turbulent section at which to take the meter readings is at mid-depth, otherwise the meter is too near the surface or too near the bottom to admit of accuracy.

Like Mr. Hoyt, the writer has summed up the measured discharges of two or more tributaries to find a good check on the discharge of the main stream. This does not prove, however, as a necessary conclusion, that the work at any of the stations is correct, as all may be, and sometimes are, affected by the same constant error.

In reply to Mr. Staniford: The writer has had only a limited experience with the current meter in salt water, in the form of a ship's log. This instrument was of the screw type, the runner being very

Mr. Groat. similar to that of the Haskell meter. With care, such an instrument will give distances, and, therefore, velocities, well within 2%, even when the sea is agitated, assuming, of course, that there is no current or surface drift.

The Haskell meter was designed by Dean Haskell when he was working in salt water, and is adapted for use in both salt and fresh water. These meters have been used in coast harbors by officers of the U. S. Engineer Corps, and the Coast and Geodetic Survey; these officers would undoubtedly be able to state how fresh- and salt-water ratings compare.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1260

HYDROLOGY OF THE PANAMA CANAL.*

By Caleb Mills Saville, M. Am. Soc. C. E.†

With Discussion by Henry L. Abbot and W. E. Fuller.

A most critical and interesting period in the history of the discharge from the Chagres River Basin has just been completed, and, on account of the wide-spread interest in the Panama Canal, it seems pertinent just now to put on record certain data bearing on the hydrology and other natural phenomena of this basin. In comparatively close sequence have followed the year of minimum run-off, the year of least dry-season run-off, and the year of maximum discharge, and these and other facts relating to the water supply of the lake seem to be of especial interest at this time. In a way, this paper may be considered as an extension of the records set forth previously by A. P. Davis, M. Am. Soc. C. E.,‡ in his reports,§ and by Gen. H. L. Abbot, Engineer Corps, U. S. A. (Retired), in his several monographs on this subject. Especial attention is directed to "Problems of the Panama Canal," by the latter author, in which may be found valuable compilations of former records, some of which are not available elsewhere. These and the deductions therefrom are the careful work of a trained hydraulician, and represent painstaking investigation.

* Presented at the meeting of March 5th, 1913.

† Formerly Assistant Engineer in Charge of Third Division, Office of Chief Engineer, Isthmian Canal Commission.

‡ Chief Engineer, U. S. Reclamation Service.

§ "Hydrology of the American Isthmus," and "Hydrology of Nicaragua." (U. S. Pub. Doc.)

The data and records used in the preparation of this paper were taken from the hydrographic and meteorological files of the Isthmian Canal Commission by the kind permission of Col. George W. Goethals, Corps of Engineers, U. S. A., M. Am. Soc. C. E., Chairman and Chief Engineer of the Isthmian Canal Commission.

Introduction.

As is generally known, the Isthmian Canal is being constructed across the Isthmus of Panama from the Caribbean Sea to the Gulf of Panama, and its location is approximately in the center of a strip of land, 10 miles wide, the use and control of which was granted in perpetuity to the United States by the Republic of Panama. This strip of land, called the "Canal Zone," stretches in a generally northwest and southeast direction, and is approximately contained between Longitude 79° 30′ and 80° 00′ west and Latitude 8° 55′ and 9° 25′ north.

Topography.

Most of that portion of the Isthmus of Panama which lies in the immediate vicinity of the Canal Zone is of little elevation above sea level, and the higher portions are either a series of more or less parallel ridges of nearly uniform height, or isolated dome-shaped hills, seemingly dropped haphazard. Along the Caribbean Coast there are many stretches of fresh swamp land, such as the Mindi Marshes southwest of Colon and reaching inland 6 or 7 miles. On both sides of the Isthmus there are also extensive salt-water shallows, mostly covered with low-growing mangroves. On the other hand, there are many bold bluffs of hard trap rock which rise abruptly from the sea, and ridges of these run back into the higher hills inland. The harbor of Porto Bello is nearly surrounded by such a barrier, and at Limon Bay, the harbor of Colon, the opposite condition appears.

On the Pacific side similar shore conditions are to be found, except that, for miles along the coast in each direction, there are no harbors which can compare with those on the Atlantic side. The main drainage of the Isthmus, of course, is dependent on the cordillera, or continental divide. About 4 miles southeast of Panama City the divide approaches within 7 miles of the Pacific shore, and for the entire distance along the southerly boundary of the Chagres Basin it is nowhere much more than 11 miles from this ocean.

The Chagres River is the largest in the Republic of Panama on the Atlantic side of the Isthmus. It has a drainage area above Gatun of 1 320 sq. miles, and between this station and the mouth of the river there are probably about 25 sq. miles of additional drainage. The Chagres discharges into the Caribbean Sea about 5 miles southwest of Toro Point, which is the northernmost limit of Canal Zone land on the westerly side of Limon Bay. Going up stream from its mouth, the course of the river is generally southeastward to Gamboa, which is about half way across the Isthmus and 35 miles from the mouth of the river. Up to this point, the rise has been very slight, the elevation of the river bed at Gamboa being only 42 ft. above sea level. The course has thus far meandered through comparatively level and oftentimes swampy bottom lands, which vary in width, to the low hills on each side. The widest stretch of this country is found in the Gatun and Trinidad marshes, lying between Gatun and Ahorca Lagarto, a swampy area about 7 miles long and from 2 to 4 miles wide, with many low, isolated hillocks scattered here and there, rising slightly above swamp level. Near Gamboa the river makes an abrupt turn almost at right angles, and the course is now generally northeastward. Its character also changes at this point, and instead of the sluggish river running through a comparatively level flood plain, with a slope of about 1¼ ft. per mile, there is now a stream, for the most part swiftly running, between banks which are the slopes of the higher hills. The Zone line is passed 40 miles above the mouth and about 5 miles above Gamboa, and the remainder of the course is in the Republic of Panama. At Alhajuela, 11 miles above Gamboa, 38½ miles above Gatun, and 46 miles above the river mouth, the bed of the stream is about 92 ft. above sea level. Twenty-four distinct rapids have been passed, and the rise has been about 50 ft. since leaving Gamboa, the average slope being about 4.6 ft. per mile between these stations. This point is approximately at proposed high-water level in Gatun Lake at its final elevation.

The climate of the Canal Zone, in a general way, has the characteristics stated by Dr. W. F. R. Phillips as belonging to the tropics:

"Unusually mild, equable, moist, warm; average temperature, 80° F.; rainfall frequent and heavy over water and over windward land exposure; nights unusually clear; afternoons cloudy; no general storms; seasons, rainy and dry, but this division only a relative one."

In regard to climatic changes and the length of time necessary for a complete cycle of fluctuation and evidence of periodicity, Willis J. Moore,* quoting Brückner, states that such a period is completed in an average of 35 years, with possible extremes of 15 years in individual cycles. Dr. W. J. S. Lockyer came to the same conclusion, using different premises from Brückner. The late George W. Rafter, M. Am. Soc. C. E.,† quoting Mr. A. R. Binnie, states that "dependence can be placed on any good record of thirty-five years' duration to give a mean rainfall correct within 2 per cent. of the truth." Mr. Rafter further states that:

"for records from twenty years to thirty-five years in length, the error may be expected to vary from 3.25 per cent. down to 2 per cent., and that for the shorter periods, of five, ten, and fifteen years, the probable extreme deviation from the mean would be 15 per cent., 8.25 per cent., and 4.75 per cent., respectively."

Station Instrumental Equipment.

Three first-class meteorological stations are in operation in the Canal Zone: Ancon, on the Pacific Coast; Culebra, in the interior on the continental divide; and Cristobal, on the Atlantic Coast. Each of these stations is equipped with the following instruments:

Standard.—Mercurial barometer, hygrometer, anemoscope, anemometer, sunshine recorder, automatic rain gauge, maximum, minimum, and standard thermometers.

Self-recording. — Thermograph, hygrograph, barograph, and meteorograph, the latter recording sunshine, rainfall, and direction and velocity of the wind.

The meteorological instruments are all of standard types, similar to those in use by the United States Weather Bureau. The Culebra station is also equipped with a maximum solar thermometer for recording the direct sun temperature, and the two coast stations have water thermographs for recording automatically the temperature of the sea water. Besides the regular meteorological stations, three wind-movement stations are maintained, at Pedro Miguel, Gamboa, and Gatun. These stations are equipped with anemometer, anemoscope, and meteorograph for recording the direction and velocity of the wind.

* "Descriptive Meteorology."

† "Water Supply and Irrigation Paper No. 80."

Twenty-five rainfall stations are maintained in the Canal Zone, nd rainfall records are also received from Chepo, Chorrera, and ocas del Toro, R. P., and from three other stations outside the Zone nd at the head-waters of representative streams.

Evaporation stations are maintained at Ancon, Rio Grande, and razos Brook Reservoirs, and at three selected locations on Gatun ake, and seismological stations at Ancon and Gatun. Continuous dal records are kept at Balboa and Cristobal.

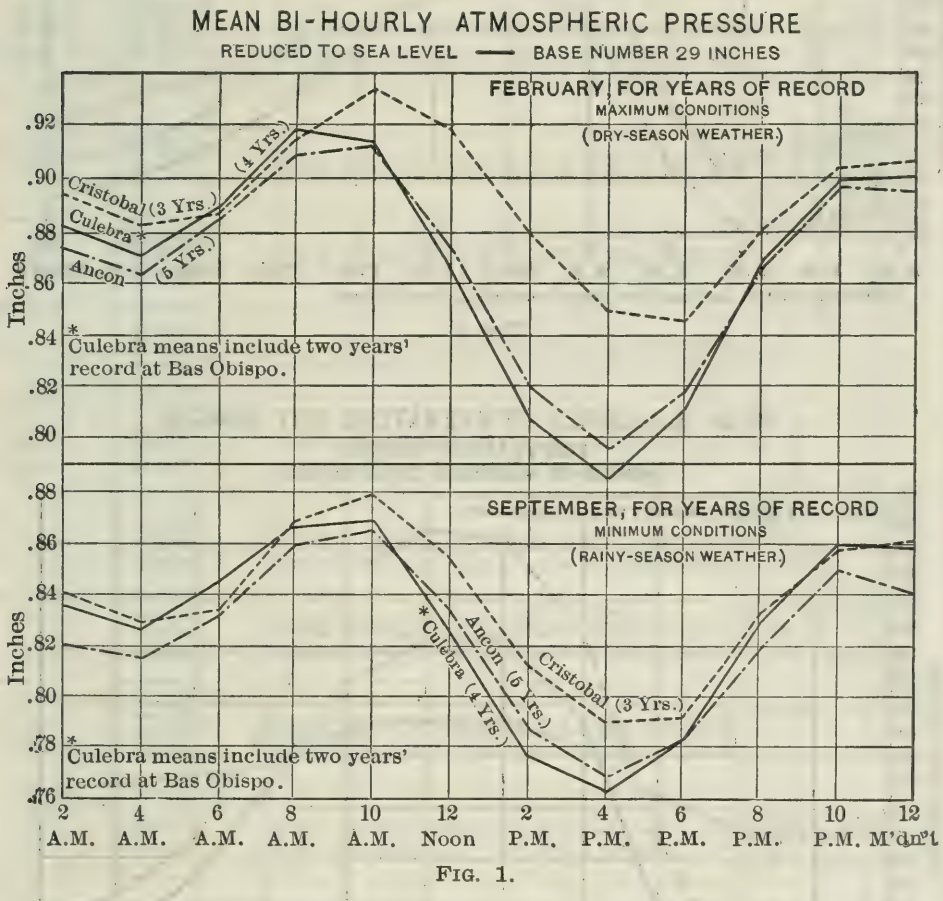


FIG. 1.

For purposes of comparison, Table 1 shows meteorological data from he three first-class stations in the Canal Zone and from Government eather reports of stations in the United States and the Philippine lands.

EVAPORATION.

Evaporation is the name of the process by which aqueous vapor is ken up from a water surface and returned to the atmosphere. By ndensation this vapor is changed back into water, and by precipita-

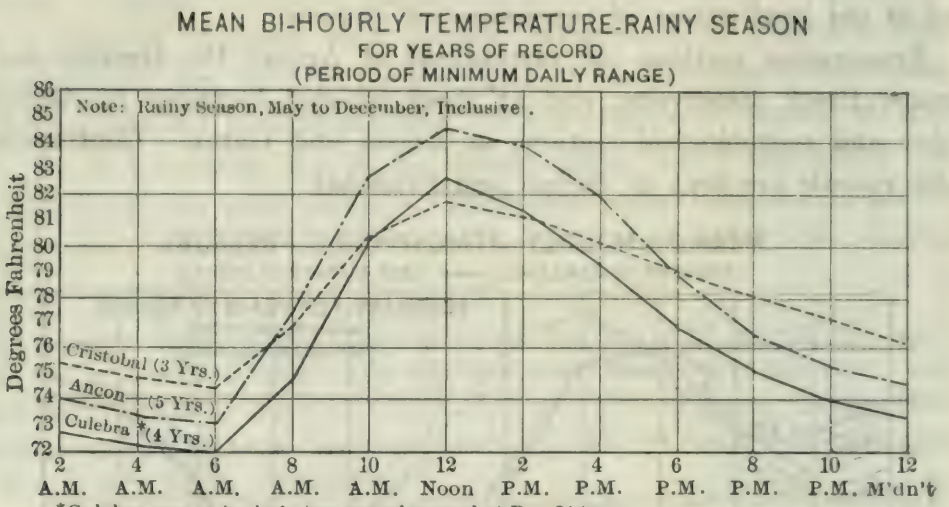

MEAN BI-HOURLY TEMPERATURE-RAINY SEASON
FOR YEARS OF RECORD
(PERIOD OF MINIMUM DAILY RANGE)
Note: Rainy Season, May to December, Inclusive.
Degrees Fahrenheit
Cristobal (3 Yrs.)
Ancon (5 Yrs.)
Culebra *(4 Yrs.)
A.M. A.M. A.M. A.M. A.M. Noon P.M. P.M. P.M. P.M. P.M. M'dn't
*Culebra means include two years' record at Bas Obispo.

FIG. 2.

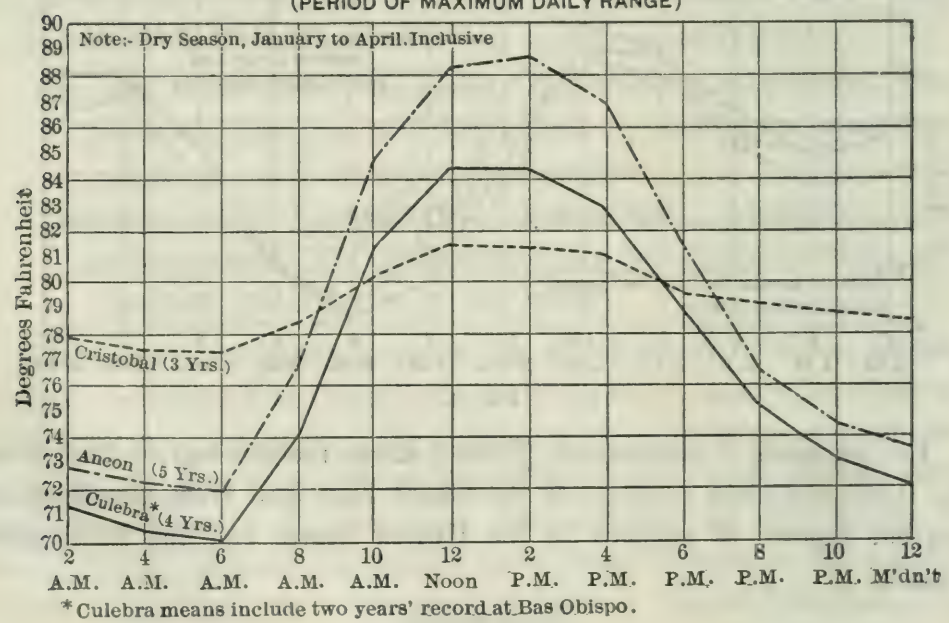

MEAN BI-HOURLY TEMPERATURE-DRY SEASON
FOR YEARS OF RECORD
(PERIOD OF MAXIMUM DAILY RANGE)
Note:- Dry Season, January to April. Inclusive
Degrees Fahrenheit
Cristobal (3 Yrs.)
Ancon (5 Yrs.)
Culebra *(4 Yrs.)
A.M. A.M. A.M. A.M. A.M. Noon P.M. P.M. P.M. P.M. P.M. M'dn't
*Culebra means include two years' record at Bas Obispo.

FIG. 3.

TABLE 1.—METEOROLOGICAL CONDITIONS AT SELECTED STATIONS.

Station.	Annual rainfall, in inches.	Average number of rainy days per annum.	Mean Relative Humidity. Percentage.			Mean Temperature, in Degrees Fahrenheit.			Mean Wind Movement. Average Hourly.			Extreme Temperature, in degrees Fahrenheit.	
			Monthly.		Annual.	Monthly.		Annual.	Monthly.		Annual.		
			Max.	Min.		Max.	Min.		Max.	Min.		Max.	Min.
Boston, Mass.	44.70	130	77.0	66.0	72.0	71.3	27.0	48.8	....	....	....	....	....
Philadelphia, Pa.	40.88	129	74.0	64.0	70.0	75.8	31.8	53.6	12.0	8.4	10.3	103	— 6
Norfolk, Va.	49.29	130	79.0	75.0	78.0	78.0	40.0	58.5	....	....	....	102	....
Jacksonville, Fla.	52.53	126	82.0	73.0	79.5	83.5	73.0	79.5	9.0	7.0	8.0	104	+ 8
Knoxville, Tenn.	48.35	135	72.0	56.0	65.0	76.2	37.5	57.4	8.0	4.7	6.2	100	— 16
Cincinnati, Ohio.	40.91	134	76.5	62.0	69.0	77.6	32.9	55.2	8.0	5.0	7.0	105	— 17
Cleveland, Ohio.	38.10	159	77.5	70.0	73.5	71.9	27.1	49.3	14.0	9.0	11.0	96	— 18
Marquette, Mich.	32.42	160	82.0	71.0	77.0	65.0	16.7	49.9	11.0	8.0	10.0	108	— 27
Peoria, Ill.	34.75	99	81.5	67.0	75.0	75.4	93.1	49.9	10.8	5.8	8.6	106	— 27
St. Paul, Minn.	27.80	114	80.0	65.0	72.5	72.0	11.9	43.9	8.8	7.1	8.1	104	— 41
Leavenworth, Kans.	34.80	65	....	....	....	78.0	25.4	53.5	....	....	....	115	— 32
Williston, Idaho.	14.86	93	77.5	60.5	69.0	68.6	6.1	39.2	11.0	8.0	9.0	107	— 49
Helena, Mont.	13.21	97	69.0	45.5	57.5	66.9	20.0	43.6	....	....	....	103	— 42
New Orleans, La.	55.63	122	79.0	74.5	72.5	81.6	51.1	63.3	8.8	6.1	7.9	102	+ 7
Fort Sill, Okla.	30.85	53	73.5	67.0	70.5	39.5	81.1	60.8	13.0	9.0	11.0	109	— 14
Galveston, Tex.	46.31	104	85.5	76.0	82.5	53.6	83.3	69.8	12.0	8.0	10.0	98	+ 8
El Paso, Tex.	9.31	47	48.0	24.0	39.5	80.6	44.1	62.9	14.0	17.0	10.0	113	+ 5
Prescott, Ariz.	17.40	47	75.0	52.0	62.0	72.7	35.1	52.7	10.0	5.0	7.0	104	— 12
Denver, Colo.	14.05	80	52.0	44.0	50.0	71.8	29.6	49.7	....	....	....	105	— 29
Salt Lake City, Utah	16.33	89	71.0	39.0	52.0	75.5	29.0	51.4	7.0	4.0	6.0	102	— 20
San Diego, Cal.	9.56	39	80.0	73.0	77.0	68.7	54.0	60.6	....	....	....	101	....
San Francisco, Cal.	22.83	71	86.0	77.5	80.5	59.3	51.0	54.9	13.5	6.9	9.7	100	+29
Portland, Ore.	42.45	164	85.0	64.0	74.0	66.6	38.7	52.8	7.0	5.0	6.0	102	— 2
CANAL ZONE.													
Ancon, (8° 57′ 25″ N.)	**71.67**	**190**	**87.6**	**73.7**	**82.9**	**81.8**	**78.8**	**80.0**	**10.0**	**5.7**	**7.3**	**96**	**63**
Culebra, (9° 02′ 23″ N.)	**90.95**	**223**	**90.0**	**76.7**	**85.4**	**79.0**	**76.9**	**78.0**	**7.3**	**4.2**	**5.4**	**94**	**59**
Cristobal, (9° 21′ 55″ N.)	**130.03**	**273**	**90.0**	**79.1**	**85.6**	**79.9**	**77.2**	**78.5**	**15.4**	**5.7**	**9.6**	**92**	**66**
PHILIPPINE ISLANDS.													
Manila, (14° 05′ N.)	76.30	....	....	....	79.5	....	....	80.2	....	....	5.7	....	....
Suriago, (9° 48′ N.)	127.86	....	....	....	85.2	....	....	79.6	....	....	....	....	...

TABLE 2.—Summary of Meteorological Data for the Canal Zone for the Years of Record.

	Ancon.	Culebra.	Cristobal.
Average number of rainy days	190	223	273
Mean annual rainfall (inches)	71.67	90.95	130.03
Maximum annual rainfall (inches)	91.42 (1901)	107.04 (1896)	183.41 (1909)
Minimum annual rainfall (inches)	45.58 (1882)	64.28 (1888)	86.54 (1884)

Annual rainfall—maximum of record on the Isthmus	237.28	Porto Bello,	1909.
Monthly rainfall—maximum of record on the Isthmus	58.17	" "	December, 1909.
	45.03	" "	November, 1909.
	42.50	Cristobal,	November, 1909.

	Ancon.	Culebra.	Cristobal.
Mean relative humidity (Percentage)			
Maximum month*	87.6 (November)	90.0 (November)	90.0 (November)
Minimum month*	73.7 (March)	76.7 (March)	79.1 (March)
Annual mean*	82.9	85.4	85.6
Mean air temperature (Degrees Fahrenheit).			
Maximum month	81.8 (April)	79.0 (April)	79.9 (April)
Minimum month	78.8 (November)	76.9 (January)	77.2 (November)
Annual mean	80.0	78.0	78.5
Mean hourly wind movement (Miles)			
Maximum month	10.0 (March)	7.3 (April)	15.4 (February)
Minimum month	5.7 (September)	4.2 (September)	5.7 (September)
Annual mean	7.3	5.4	9.6
Extreme air temperature (Degrees Fahrenheit)			
Maximum	96.0 (March 20th, 1908)	94.0 (April 15th, 1909)	92.0 (June 3d, 1909)
Minimum	63.0 (January 27th, 1910)	59.0 (February 9th, 1907)	66.0 (December 3d, 1909)

* Mean of bi-hourly values.
Rainfall records cover 13 years at Ancon, 20 years at Culebra, and 40 years at Cristobal.
Humidity, temperature, and wind records cover 5 years at Ancon, 4 years at Culebra, and 3 years at Cristobal.
Culebra records include 2 years at Bas Obispo.

tion the water is again returned to the earth, completing the so-called "meteorological cycle." The prevailing winds, constantly blowing over large bodies of water, keep the saturated vapor always moving on, and dry air behind it flows in, taking up in turn its load of moisture which it carries to land. When the elevation of the land is only a little above water level, there is formed a vast blanket of air thoroughly saturated and ready for any disturbance to cause it to drop its burden in the form of rain. As the air moves away from the water surface, it meets with more and more obstructions, in the form of hills and ridges, and by them is deflected upward into the colder areas. There, becoming condensed, the vapor falls to earth in the form of rain. In the regions of steadily prevailing winds like the trades, the windward slope of the mountain ranges is the one of more copious rainfall.

The laws of evaporation are probably very complex, and although much study and investigation have been given to the phenomena, no satisfactory conclusions have been reached as yet which are applicable to all conditions. The principal factors influencing evaporation are wind, temperature, and vapor pressure. In a region like the Canal Zone, where atmospheric temperature and pressure are exceedingly uniform, the effect of the wind is of paramount importance in causing variation in the rate of evaporation. The diagram, Fig. 5, shows the relation between these factors for the years of record at the Brazos Brook station. The parallelism of the evaporation and wind velocity curves is especially noticeable, as is also the inverse relationship of these curves and the lines of monthly rainfall. The air temperature curve, which is that of the monthly mean, although very uniform, still indicates a certain parallelism with the evaporation curve, while the opposite is shown in the curve of vapor pressure. The diagram, Fig. 8, shows graphically the number of days on which the evaporation at Rio Grande Reservoir and at Brazos Brook Reservoir have exceeded a stated quantity. From these curves it appears that the mean daily evaporation for dry-season conditions is about $\frac{1}{6}$ in. per day, and that this rate is only exceeded for about 80 days. This rate means a lowering of the lake by about 1.7 ft. during the entire dry season, with no allowance for increase by rainfall on the surface. The high rates are recorded for short periods—$\frac{1}{5}$ in. and more for 40 days, and $\frac{1}{4}$ in. per

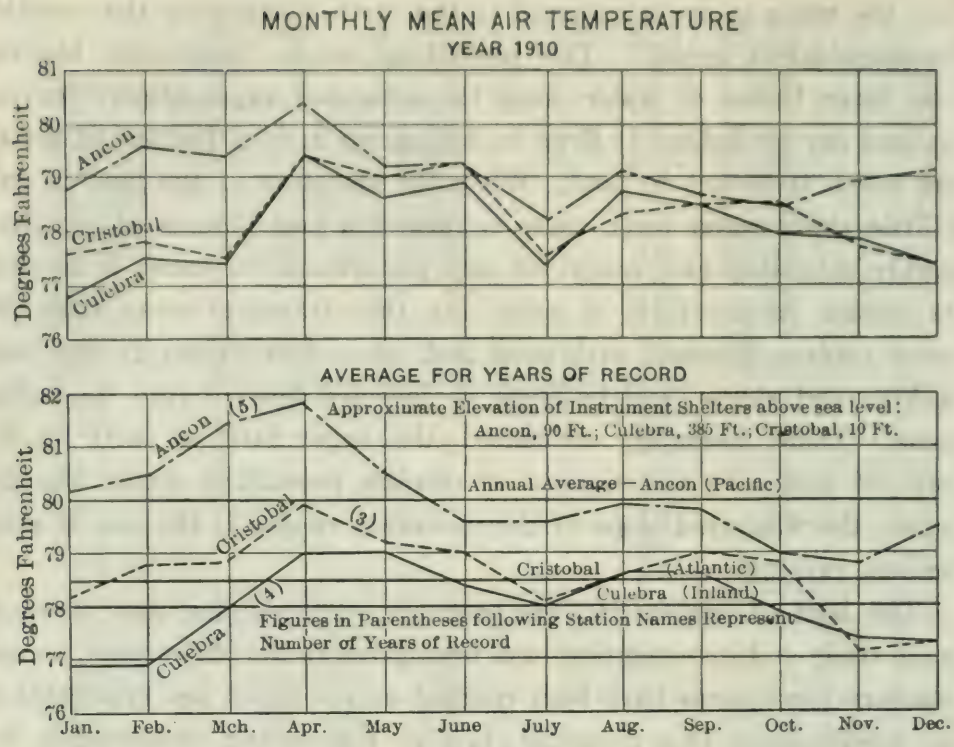

MONTHLY MEAN AIR TEMPERATURE
YEAR 1910
Degrees Fahrenheit
81
80
79
78
77
76
Ancon
Cristobal
Culebra
AVERAGE FOR YEARS OF RECORD
Approximate Elevation of Instrument Shelters above sea level:
Ancon, 90 Ft.; Culebra, 385 Ft.; Cristobal, 10 Ft.
Annual Average — Ancon (Pacific)
Cristobal (Atlantic)
Culebra (Inland)
Figures in Parentheses following Station Names Represent Number of Years of Record
(5)
(3)
(4)
Jan. Feb. Mch. Apr. May June July Aug. Sep. Oct. Nov. Dec.

FIG. 4.

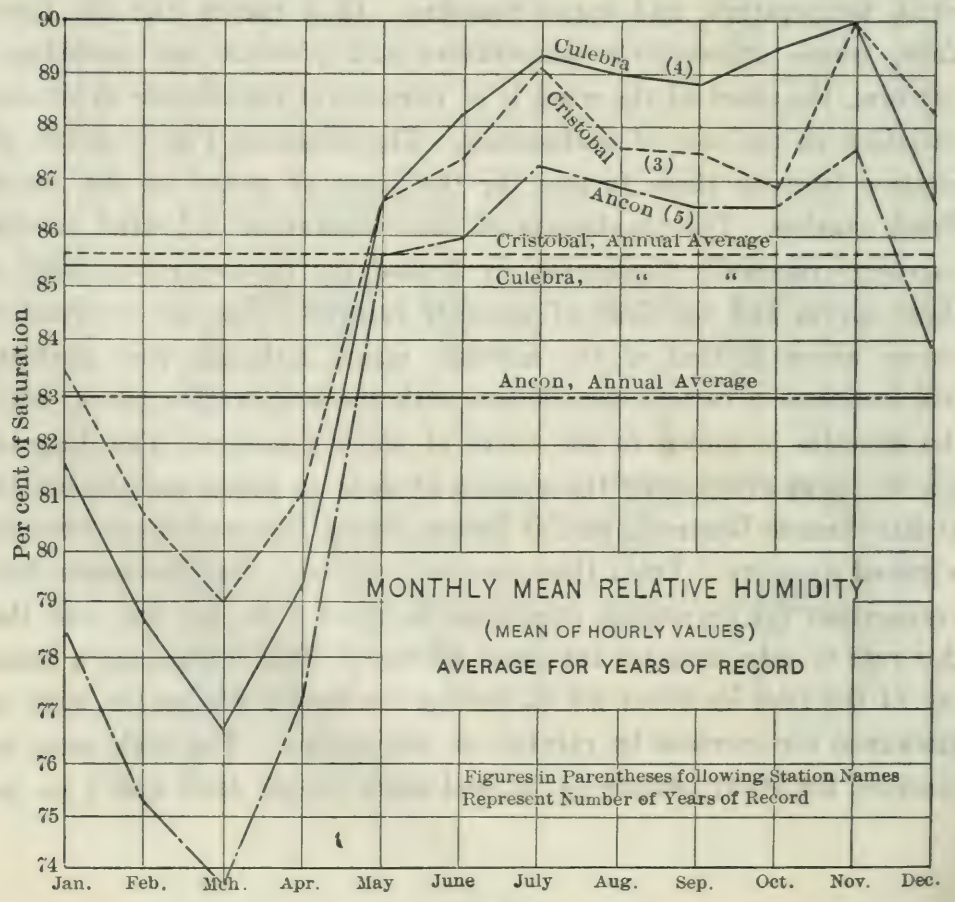

Per cent of Saturation
90
89
88
87
86
85
84
83
82
81
80
79
78
77
76
75
74
Culebra (4)
Cristobal (3)
Ancon (5)
Cristobal, Annual Average
Culebra, " "
Ancon, Annual Average
MONTHLY MEAN RELATIVE HUMIDITY
(MEAN OF HOURLY VALUES)
AVERAGE FOR YEARS OF RECORD
Figures in Parentheses following Station Names
Represent Number of Years of Record
Jan. Feb. Mch. Apr. May June July Aug. Sep. Oct. Nov. Dec.

FIG. 5.

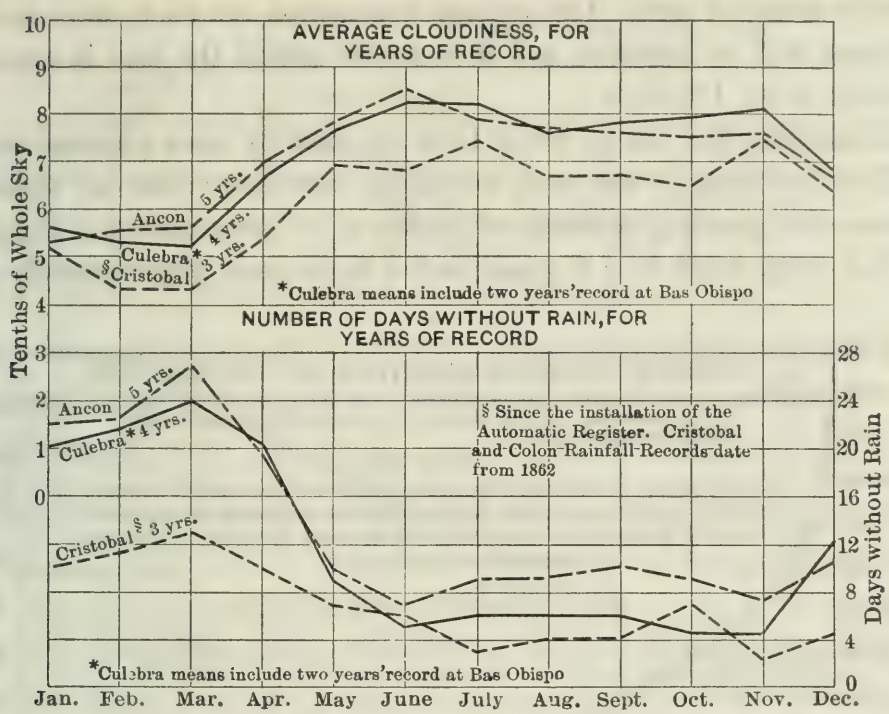

AVERAGE CLOUDINESS, FOR YEARS OF RECORD
Tenths of Whole Sky
Ancon
Culebra *
§Cristobal
5 yrs.
4 yrs.
3 yrs.
*Culebra means include two years' record at Bas Obispo
NUMBER OF DAYS WITHOUT RAIN, FOR YEARS OF RECORD
Days without Rain
§ Since the installation of the Automatic Register. Cristobal and Colon Rainfall Records date from 1862
*Culebra means include two years' record at Bas Obispo
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 6.

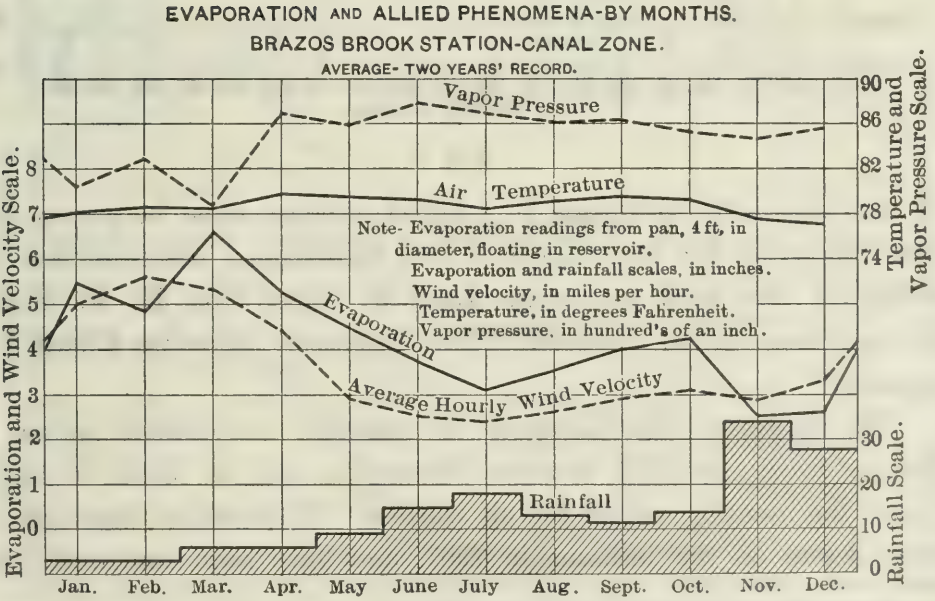

EVAPORATION AND ALLIED PHENOMENA-BY MONTHS.
BRAZOS BROOK STATION-CANAL ZONE.
AVERAGE- TWO YEARS' RECORD.
Vapor Pressure
Air Temperature
Note- Evaporation readings from pan, 4 ft, in diameter, floating in reservoir.
Evaporation and rainfall scales, in inches.
Wind velocity, in miles per hour.
Temperature, in degrees Fahrenheit.
Vapor pressure, in hundred's of an inch.
Evaporation
Average Hourly Wind Velocity
Rainfall
Evaporation and Wind Velocity Scale.
Temperature and Vapor Pressure Scale.
Rainfall Scale.
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 7.

day for about 10 days. The average evaporation for the 8 rainy months is about 0.11 in. per day, and during this period the rate is exceeded on only about 120 days.

Regarding the loss of water from the lake by trees growing in the shallow portions, it has been found by experience that all standing timber now growing on lands which are to be submerged will be killed by the water after 2 or 3 years, and a large area, approximately that

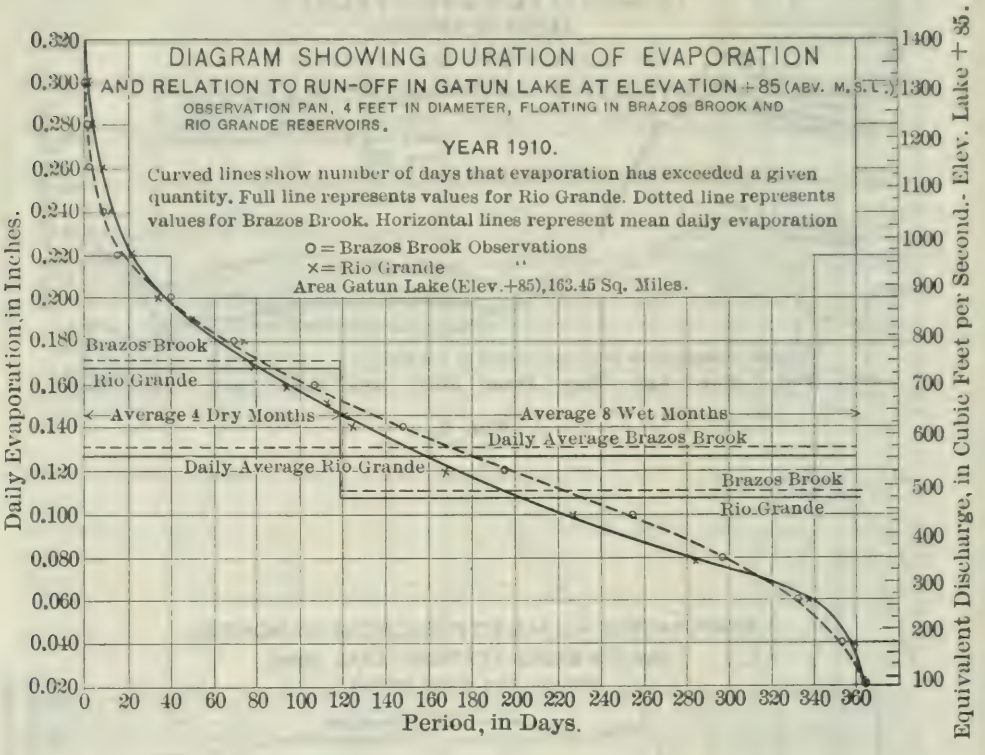


FIG. 8.

of the lake itself, will then be relieved of what must be a very large draft necessary for the support of this vegetation. Attention was first called to this gain to Gatun Lake by Lieut.-Col. H. F. Hodges, M. Am. Soc. C. E., Assistant Chief Engineer, Isthmian Canal Commission.*

The diagram, Fig. 9, shows the monthly evaporation values for Brazos Brook and Rio Grande Reservoirs, and in Table 3 may be found the monthly values for all stations. Evaporation measurements were begun on the Isthmus in 1907, and have been continued since.

* In his discussion of the paper entitled "Water Supply for the Lock Canal at Panama," by Julio F. Sorzano, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVII, p. 97.

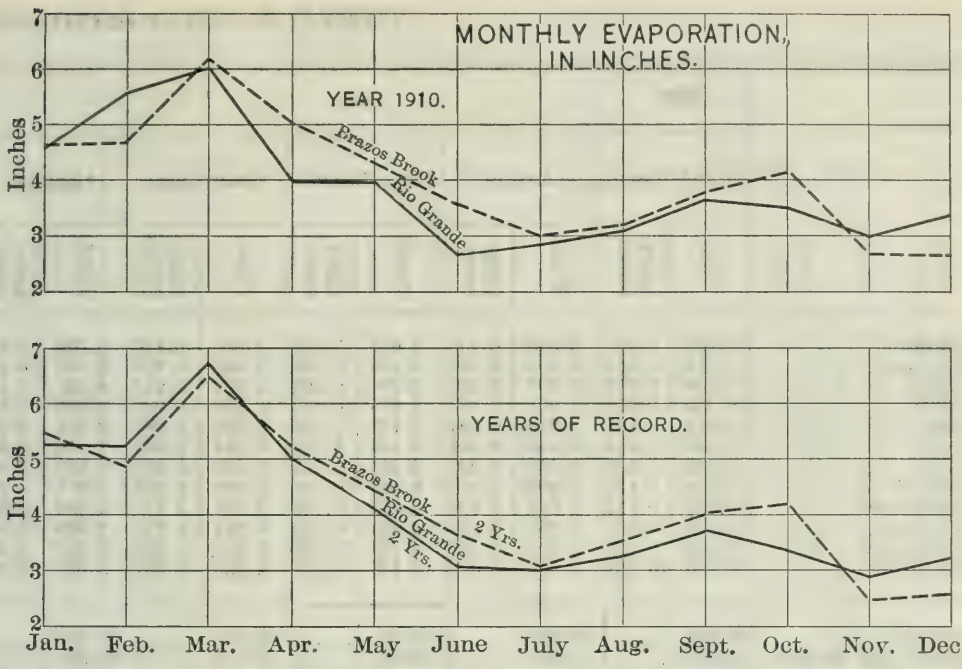

MONTHLY EVAPORATION, IN INCHES.
YEAR 1910.
Brazos Brook
Rio Grande
Inches
YEARS OF RECORD.
Brazos Brook
2 Yrs.
Rio Grande
2 Yrs.
Inches
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 9.

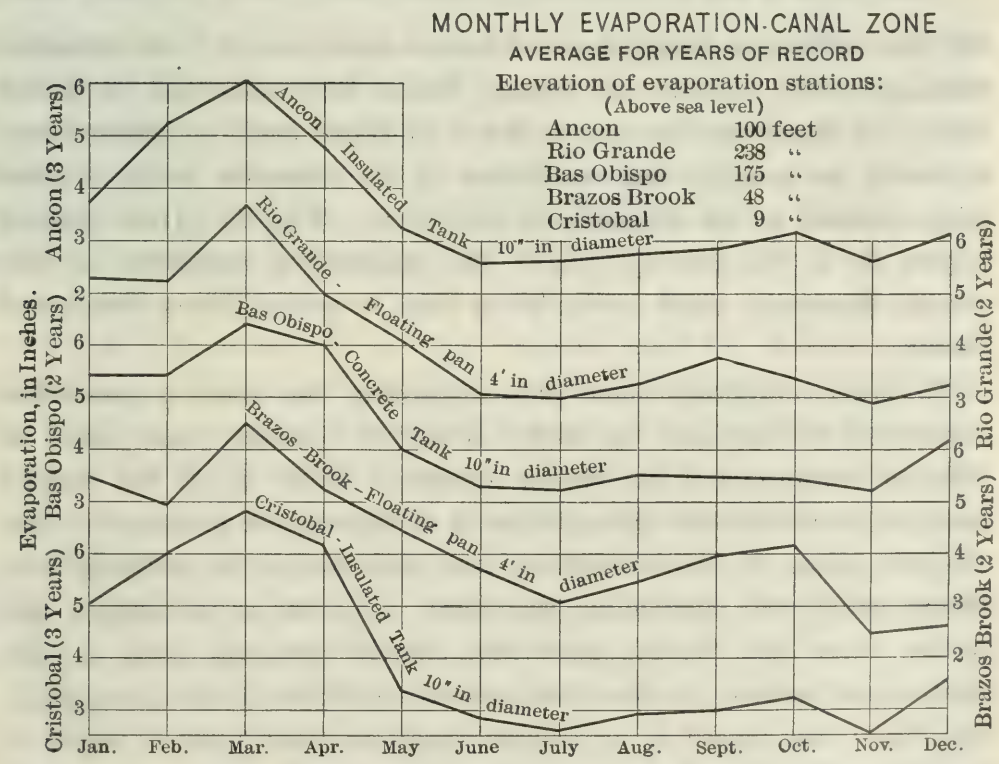

MONTHLY EVAPORATION-CANAL ZONE
AVERAGE FOR YEARS OF RECORD
Elevation of evaporation stations:
(Above sea level)
Ancon 100 feet
Rio Grande 238 "
Bas Obispo 175 "
Brazos Brook 48 "
Cristobal 9 "
Ancon Insulated Tank 10" in diameter
Rio Grande - Floating pan 4' in diameter
Bas Obispo - Concrete Tank 10" in diameter
Brazos Brook - Floating pan 4' in diameter
Cristobal - Insulated Tank 10" in diameter
Evaporation, in Inches.
Ancon (3 Years)
Bas Obispo (2 Years)
Cristobal (3 Years)
Rio Grande (2 Years)
Brazos Brook (2 Years)
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 10.

TABLE 3.—EVAPORATION IN

	1907.		1908.							
	‡Bas Obispo.		†Ancon.		‡Bas Obispo.		†Cristobal.		†Ancon.	
	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.
January	5.175	0.167	3.348	0.108	5.617	0.181	4.868	0.157	3.348	0.108
February	5.072	0.181	6.557	0.226	5.729	0.198	7.958	0.274	3.728	0.133
March	6.538	0.211	6.997	0.226	6.290	0.203	7.557	0.244	4.931	0.159
April	6.486	0.216	5.921	0.197	5.475	0.182	6.930	0.231	3.883	0.129
May	4.681	0.151	3.219	0.104	3.175	0.102	2.570	0.083	2.451	0.079
June	3.125	0.104	3.046	0.101	3.415	0.114	3.184	0.106	1.916	0.064
July	3.152	0.102	3.017	0.097	3.250	0.105	2.822	0.091	2.080	0.067
August	3.582	0.116	3.203	0.103	3.425	0.110	3.192	0.103	2.271	0.073
September	3.358	0.112	3.049	0.102	3.635	0.121	3.327	0.111	2.484	0.083
October	2.938	0.095	3.256	0.105	3.875	0.125	3.324	0.107	2.826	0.091
November	3.599	0.120	2.414	0.080	2.730	0.091	2.700	0.090	2.332	0.078
December	4.896	0.158	2.942	0.095	3.445	0.111	4.056	0.131	3.020	0.097
Annual	52.602	.....	46.996	.....	50.061	.....	52.488	.....	35.270	.....
Sums, Apr. to Dec.	35.817	.....	30.067	.....	32.425	.....	32.105	.....	23.263	.....
Mean, 12 months	4.384	0.144	3.914	0.129	4.172	0.137	4.374	0.144	2.939	0.097
Mean, 9 months	3.980	0.130	3.341	0.109	3.603	0.118	3.567	0.117	2.585	0.085

* Tank floating in lake; 4 ft. diameter. † Insulated tank, protected from direct rays of

The best values are being obtained from copper pans, 4 ft. in diameter, floating in Rio Grande and Brazos Brook Reservoirs and in Gatun Lake. At these stations the evidence is being made to approximate as nearly as possible the conditions of an extensive water surface fairly exposed to all atmospheric conditions. Fig. 11 (from Annual Report, I. C. C., 1910-11) shows the evaporation apparatus at Rio Grande Reservoir, which is similar to those in use at Brazos Brook and Gatun.

In order to obtain some data regarding the relative quantities evaporated by day and by night, series of readings were taken at different stations, and the results appear in Table 5. It has seemed desirable to obtain some information as to the relative evaporation that will take place in various parts of the lake due to the differing conditions which will exist along the shore. In order to investigate this matter, three 4-ft. floating pans were placed in Gatun Lake in such locations as seemed to cover the various conditions to be anticipated. Pan No. 1 was placed in an exposed location about 1 000 ft. south of the dam and an equal distance from each shore or other obstruction.

THE CANAL ZONE, IN INCHES.

1909.						1910.							
*Rio Grande.		*Brazos Brook.		†Cristobal.		†Ancon.		*Rio Grande.		*Brazos Brook.		*Cristobal.	
Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.
.....	.....	.....	.....	4.868	0.157	4.354	0.140	4.612	0.149	4.622	0.149	5.257	0.170
.....	.....	.....	.....	5.353	0.191	5.350	0.191	5.529	0.197	4.668	0.167	4.574	0.163
.....	.....	.....	.....	6.445	0.208	6.190	0.200	6.003	0.194	6.151	0.198	6.500	0.210
5.960	0.199	5.366	0.179	6.468	0.216	4.379	0.146	3.986	0.133	5.025	0.168	5.209	0.174
4.205	0.136	4.597	0.148	3.868	0.125	3.378	0.109	3.916	0.126	4.304	0.139	3.493	0.113
3.417	0.114	3.806	0.127	2.498	0.083	2.666	0.089	2.654	0.088	3.516	0.117	2.703	0.090
3.117	0.101	3.042	0.098	2.445	0.079	2.740	0.088	2.846	0.092	3.014	0.097	2.359	0.076
3.353	0.108	3.760	0.121	2.636	0.085	2.866	0.092	3.096	0.100	3.189	0.103	2.852	0.092
3.768	0.126	4.169	0.139	2.727	0.091	3.078	0.103	3.677	0.123	3.804	0.127	2.791	0.093
3.094	0.100	4.168	0.134	3.027	0.098	3.436	0.111	3.577	0.115	4.177	0.135	3.343	0.108
2.713	0.090	2.152	0.072	1.950	0.065	3.132	0.104	2.999	0.100	2.718	0.091	2.817	0.094
2.993	0.096	2.379	0.077	2.746	0.089	3.528	0.114	3.442	0.111	2.746	0.089	3.732	0.120
.....	.....	.....	.....	45.031	.....	45.097	.....	46.337	.....	47.934	.....	45.630	.....
32.620	.....	33.439	.....	28.365	.....	29.203	.....	30.193	.....	32.493	.....	29.299	.....
.....	.....	.....	.....	3.753	0.124	3.758	0.124	3.861	0.127	3.994	0.131	3.802	0.125
3.624	0.119	3.715	0.122	3.152	0.103	3.245	0.106	3.355	0.110	3.610	0.118	3.255	0.107

sun; 10 in. diameter. ‡ Concrete tank, 12½ ft. diameter, 5 ft. deep.

This exposure was intended to represent ordinary conditions on the open lake. Pan No. 2 was placed among the trees of what had been a considerable forest, but which was now partly submerged by the rising waters of the lake. It is located about 500 ft. in from the edge of the forest, several trees being felled in order that the records might not be affected by rain dropping from the leaves. Pan No. 3 was placed in the center of an area of tall water grass, about ½ acre in extent. Such a growth as this will be very frequent along the shallow portions of the shore. In the last two exposures it appears that the action of the wind is a very great factor in determining the quantity of evaporation. It also demonstrates that the action of the shore growth will be that of a wind-break, and that the evaporation figures based on data collected from exposed locations are on the safe side, that is, probably they will give too great an evaporation. Table 4 shows the results obtained since the installation of the pans.

The equipments at Rio Grande, Brazos Brook, and Gatun consist of the following measuring instruments:

A copper pan, 4 ft. in diameter and 10 in. deep, floating in the

water, for evaporation; an anemometer, for wind movement directly over the evaporation pan; a rain gauge, for rainfall; a thermometer, for water temperatures, and maximum and minimum thermometers, exposed in an instrument shelter of the regular type located on the shore close by, for recording the air temperatures.

The evaporation pan is set in a light wooden frame, properly buoyed, which serves to keep the top of the pan above the surface of the water in the lake, as well as to prevent occasional waves from splashing over into the pan. The pan rests with its bottom and sides submerged in the water, thus insuring that the temperature of the water within the pan at all times will be very close to that of the water at the surface. The float is located at a considerable distance from shore, for the purpose of getting an exposure which represents actual average conditions.

A sharp-pointed, copper-wire index is set in the center of the pan and inside a perforated copper cylinder still-box which serves to reduce the wave motion of the surrounding water, and thus permits more accurate readings of the daily evaporation. The index point is 4 in. below the top of the pan, and it follows that the usual height of the water in the pan is also approximately 4 in. below the top. This arrangement is necessary to prevent the overflow of the pan during heavy rains.

TABLE 4.—Evaporation Records for Pans Floating in Gatun Lake.

Monthly Values.

Month.	Pan No. 1.			Pan No. 2.			Pan No. 3.		
	Rainfall, in inches.	Wind average, in miles per hour.	Evaporation, in inches.	Rainfall, in inches.	Wind average, in miles per hour.	Evaporation, in inches.	Rainfall, in inches.	Wind average, in miles per hour.	Evaporation, in inches.
1911.									
May*	15.33	3.93	2.12	14.21	1.47	1.75	14.41	....	1.83
June	16.48	3.86	2.80	17.65	0.97	2.41	17.89	....	2.48
July	7.20	5.90	4.56	7.31	1.47	3.28	7.33	....	2.78
August	8.19	4.74	4.41	6.77	0.86	3.07	7.35	....	3.37
September	4.83	5.02	4.58	7.10	1.04	2.63	6.96	....	3.13
October	16.85	4.11	3.53	16.62	0.68	2.77	15.21	....	3.00

* No record for the first 10 days of May.

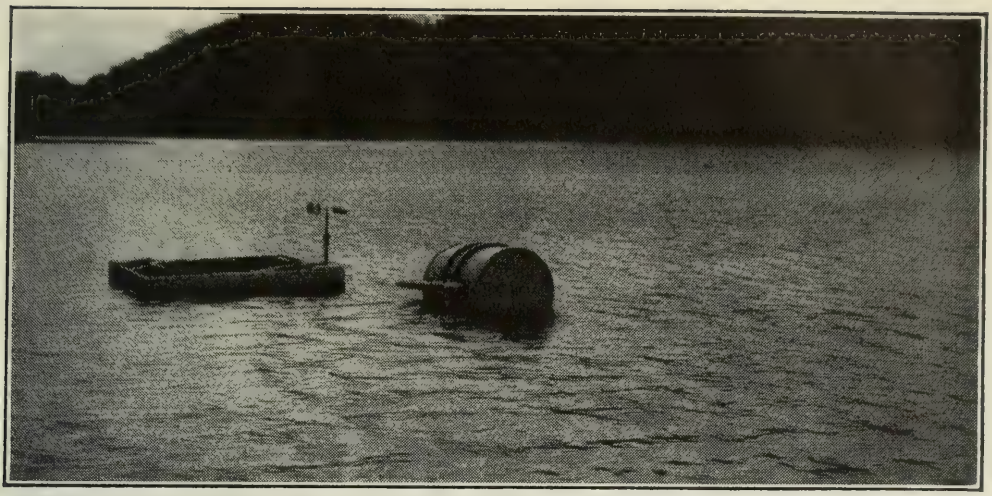


FIG. 11.—EVAPORATION GAUGE IN LAKE.

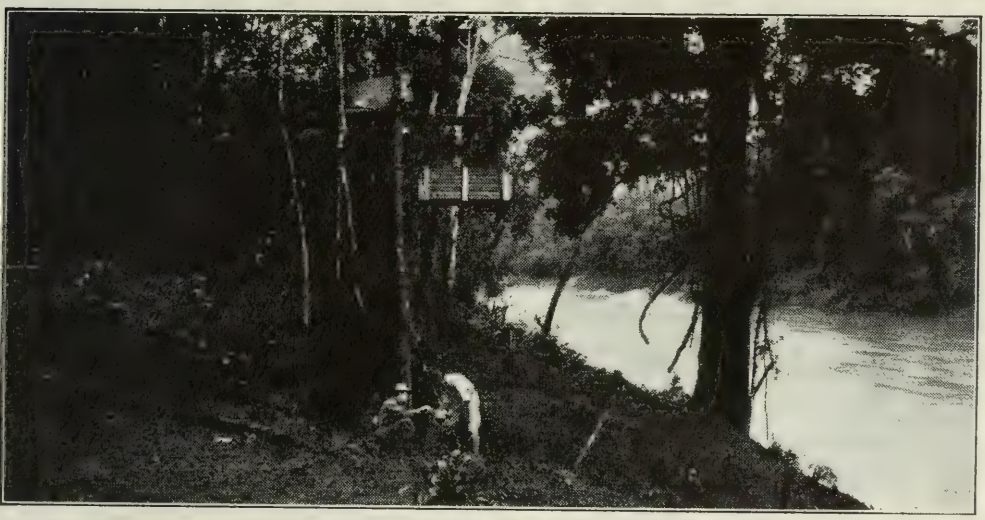


FIG. 12.—CABLE ANCHORAGE AND GAUGING CAR, ALHAJUELA.

FIG. 13.—FLUVIOGRAPH, ALHAJUELA STATION.

TABLE 5.—Evaporation in Canal Zone. Comparative Values for Day and Night.

Month.	1908.*						1909.				1911.*	
	Ancon.		Bas Obispo.		Cristobal.		Ancon.		Cristobal.		Cristobal.	
	Day.	Night.	Day.	Night.	Day.	Night.	Day.	Night.	Day.	Night.	Day.	Night.
January	.....	.....	.....	.....	.....	.....	1.755	1.593	2.754	2.114	4.600	3.565
February	5.062	1.495	3.605	2.124	4.200	3.758	2.134	1.594	2.916	2.437	3.763	3.341
March	5.566	1.431	3.919	2.371	4.402	3.155	2.693	2.238	3.660	2.785	4.810	4.275
April	4.404	1.517	3.630	1.845	3.999	2.931	2.108	1.775	3.594	2.873	3.152	2.247
May	2.001	1.218	1.715	1.460	1.706	0.864	1.317	1.134	2.268	1.600	1.973	1.323
June	2.034	1.012	1.955	1.460	1.878	1.306	0.941	0.975	1.332	1.166	1.674	1.276
July	1.900	1.117	1.670	1.580	1.584	1.238	1.078	1.002	1.252	1.193	.....	.....
August	2.175	1.028	1.702	1.723	1.985	1.207	1.241	1.030	1.467	1.169	.....	.....
September	1.924	1.125	2.178	1.457	2.096	1.231	1.425	1.059	1.706	1.021	.....	.....
October	2.051	1.205	2.467	1.408	2.016	1.308	1.862	0.964	1.784	1.243	.....	.....
November	1.362	1.052	1.555	1.175	1.572	1.128	1.395	0.937	1.129	0.821	.....	.....
December	1.337	1.605	2.040	1.405	2.252	1.804	1.775	1.245	1.484	1.262	.....	.....
Total	29.816	13.805	26.436	18.008	27.690	19.930	19.724	15.546	25.346	19.684	19.972	16.027
Percentage	68	32	60	40	58	42	56	44	56	44	55	45

* 11 months, 1908; Cristobal, 6 months, 1911.
Readings taken at 8.00 A. M. and 8.00 P. M. daily.
Mean of all observations, 59% day and 41% night.

The pan is set by bringing the surface of the water exactly to the level of the point of the copper index. When the evaporation measurements are made, 24 hours later, if there has been no rainfall, the surface of the water in the pan will be found below the index point. The quantity of water necessary to raise the water surface to the level of the index point is carefully measured and poured into the pan. This quantity of water, expressed in thousandths of an inch over the surface of the evaporation pan, equals the evaporation for the previous 24 hours. When the rainfall during the day has exceeded the evaporation, the surface of the water in the pan will be found above the point of the copper index. The excess water must be removed from the pan and measured. In this case the difference between the rainfall and the quantity of water removed from the pan, expressed in thousandths of an inch, equals the evaporation for the previous 24 hours. When rain occurs during the day, but is less than the evaporation, the sum of the rainfall and the quantity of water poured into the pan, expressed in thousandths of an inch, equals the evaporation. The water poured into or removed from the evaporation pan is measured in a copper measuring tube having exactly $\frac{1}{100}$ of the cross-section

area of the evaporation pan. Depths are measured with the regular rainfall measuring stick, graduated to inches and tenths; $\frac{1}{10}$ in. in the measuring tube represents $\frac{1}{1000}$ in. over the surface of the evaporation pan.

Table 5 shows the quantities of evaporation, by day and by night, that have been recorded at the several stations on the Canal Zone, together with the relative percentages. The mean of the entire series is 59% by day and 41% by night.

Besides the direct loss due to the run-off by evaporation from water surfaces, a large percentage of the rainfall is still unaccounted for, which is aptly called "retention" by Professor D. W. Mead,* M. Am. Soc. C. E.; part of this is used by vegetation and transpired by the leaves, part is retained in the ground and returned to the stream at periods of low rainfall, and a portion is permanently lost to the watershed either by deep percolation or, as is sometimes the case, by meeting a permeable stratum which conducts it to another basin. This latter cause, however, is usually relatively unimportant, but should not be overlooked in the examination of a water supply. In the Gatun Lake Basin, due to the practically impervious argillaceous sandstone and other rocks of similar character which appear to underlie the entire lake area, permanent loss of water to the basin is probably too small for any consideration, and is neglected hereafter in this paper. The following shows the portion of the rainfall unaccounted for in certain northern streams:

Rivers.	Portion of rainfall unaccounted for.
Hudson, N. Y.	52 per cent.
Sudbury, Mass.	49 " "
Croton, N. Y.	46 " "
Wisconsin, Wis.	65 " "
Chippewa, "	50 " "
Rock, "	21 " "

In his treatise on water supply engineering, the late J. T. Fanning, M. Am. Soc. C. E., has given values from investigations at Emdrup, Denmark, from which the following is deduced:

* "Flow of Streams and the Factors that Modify It," *Bulletin No. 425*, Univ. of Wisconsin.

	Mean evaporation.	Ratio to water evaporation.
Water surfaces, mean evaporation (annual)	27.9 in.	1.00
Short grass	30.1 "	1.08
Long grass	44.0 "	1.58
From Bradmore's "Hydrology."		
Lancashire, England, Lat. 53° 30′ N., from earth	25.7 "	....
Cumberland, England, Lat. 54° 34′ N., from earth	29.2 "	....

Investigations by Risler concerning the quantity of water required or different crops* are tabulated below, and give the daily consumption of water for various crops:

Lucerne grass	from	0.134 to 0.267	in.
Meadow "	"	0.122 " 0.287	"
Oats	"	0.140 " 0.193	"
Indian corn	"	0.110 " 1.570	"
Clover	"	0.140 "	"
Vineyard	"	0.035 " 0.031	"
Wheat	"	0.106 " 0.110	"
Rye	"	0.091 "	"
Potatoes	"	0.038 " 0.055	"
Oak trees	"	0.038 " 0.030	"
Fir trees	"	0.020 " 0.043	"

Mr. Tweeddale states as his opinion that cereals and grasses, from seed time to harvest, will take up, respectively, from 15 to 37 in.

In his report on water supply, Mr. C. C. Vermeule has used the following factors for the water required by various crops in one growing month:

Crop.	Inches.
Forest (oak and chestnut)	1.2
Wheat, rye, oats, etc.	3.5
Indian corn	4.5
Potatoes, etc.	1.2
Long grass	6.0

* Quoted by W. Tweeddale, Kansas State Board of Agriculture, December 31st, 1889.

Crop.	Inches.
Short grass	5.0
Orchards	3.0
Fallow lands, etc	4.0

In investigation work at Pomona, Cal.,* the following duty has been required of the water used on various crops for the average year (1906-09):

Citrus fruits (oranges and lemons)	2.94 ft.
Deciduous fruits and miscellaneous crops	2.58 "
Alfalfa	4.6 "

Although these figures do not show the quantity of water that similar crops would use were they entirely dependent on natural supply, they do indicate the relative quantities that such crops demand for the best conditions, and the quantity they might use if it were available. The depth of water used on a 21½-acre apple orchard in Wenatchee, Wash.,† was found to be 23 in. during the year, the trees being 7 years old and producing heavily. Mr. Samuel Fortier‡ states that the results of experiments have shown that, where water is applied to the surface of orchard soils, the evaporation is very great as long as the top layer remains moist. Even in light irrigation, the loss in 48 hours after the water is put on may amount to from 10 to 20% of the volume applied. Mr. W. W. McLaughlin,§ in discussing the irrigation of grain, states that it is always safe to assume that the ranker the growth of straw the greater will be the quantity of water required. He also says that the total depth of water applied to grain crops on the older lands will vary from 1 to 8 ft., depending on the porosity of the soil and the climate of the place, average depths of 2 and 3 ft. representing the quantities required, respectively, in cool and warm climates. Field peas‖ in an average irrigation season of 42 days are shown to have required about 4.7 in., or about 0.112 in. per day.

The foregoing figures, though not exact, and applying to crops and conditions elsewhere and under different climatic conditions, are nevertheless instructive in obtaining some idea of the quantity of water which must be required by the rank and lush vegetation of this

* *Bulletin 236*, Office of Exper. Station, U. S. Dept. of Agriculture, p. 87.
† *Farmers' Bulletin 404*, U. S. Dept. of Agriculture, p. 26.
‡ *Farmers' Bulletin 404*, U. S. Dept. of Agriculture, p. 28.
§ *Farmers' Bulletin 399*, U. S. Dept. of Agriculture, p. 18.
‖ *Bulletin 72*, Wyoming Experiment Station, p. 7.

ropical country. With all its heavy precipitation, were it not for the teep hillsides, and consequent quick discharge of the waters, it is doubtful if the ratio of run-off to rainfall might not even be less here than n countries where the rainfall is smaller but the vegetation less uxuriant.

E. Oppokov* states that marshes are a detriment to the drainage rea of a water supply system. It is ordinarily thought that such areas re valuable for the storage of water in the rainy season, and that luring the dry period they allow of a gradual discharge, which increases he stream flow. Having studied the features of the Dnieper and djoining rivers, Mr. Oppokov believes that his investigations prove here is intensified evaporation from marsh areas in periods of drought, vhich not infrequently results in the total drying out of the marsh. These areas then act as a great sponge for the absorption of subsequent precipitation and seepage waters from the uplands, and cause decreased returns of the waters to the rivers, the run-off being actually essened thereby.

In the case of the Gatun Lake water-shed, there will probably be n increase in run-off when the swamp areas are eliminated by submergence. Under present conditions, these areas annually absorb arge quantities of the rainfall to make up for the depletion caused by evaporation and plant necessity. After submergence, the trees nd other vegetation will die off, and the ground will become saturated nce for all. Subsequent rainfall will then be available for lake purposes. Such conditions will be especially appreciated during periods f light rainfall, when the demands of vegetation are greatest.

The fact that a considerable rainfall may occur with little run-off rom a basin overgrown with vegetation, and that the daily evaporation rom a water surface is relatively small in comparison with the precipitation, is an indication that the available run-off will be increased. For river regulation and the prevention of floods, this condition might present an aspect entirely different from what it has when applied to the conservation of the water. The well-sustained run-off from the area bove Alhajuela is due, not only to its large rainfall, but probably also o the absence of swamps with their shallow storage, and the presence f an extensive limestone formation. This latter undoubtedly allows he deep storage of large quantities of water during the rainy season,

* *Russ. Jour. Expr.*, Landau, 1910, pp. 369-372.

which, as the dry season proceeds, is given off for the benefit of river discharge. The swamp and the limestone storage, although both classed as underground, to distinguish them from surface water, are nevertheless entirely distinct in their tendencies. The former is shallow and readily affected by the agencies producing evaporation, including plant demands; the latter, deep and far removed from the surface losses, is all available for stream flow, if it can but find an outlet.

An excellent example of this latter condition is found in a limestone cliff overhanging the Chagres River above Alhajuela. Here is a nearly perpendicular wall, more than 100 ft. above the river surface, and having a length of more than 500 ft. This whole surface, more or less covered with moss and clinging vegetation, is constantly and copiously dripping with water during the entire year. The contour surveys and explorations have shown that there are no extensive swamp areas in the Chagres River Basin above Elevation + 60, and it seems fair to presume that, when the lake has reached its final elevation, hydrographic conditions similar to those now observed for the area above Alhajuela will obtain elsewhere, modified by local conditions of rainfall.

The geological and topographical investigations which have been made in the lake area and its drainage basin seem to give assurance of exceedingly favorable conditions for the collection of the water supply and its conservation. The lower part of the Chagres Basin, as has been stated, is underlaid by argillaceous sandstone, and this is the principal constituent in the ridges bordering the lake. Nearly the entire bottom of the lake, up to about Elevation + 30, is covered with a deposit of alluvium, which is also practically impervious. This cover, about 51 miles in extent, and of varying thicknesses up to several hundred feet, presents an additional safeguard against bottom seepage, and at the place where, if needed at all, it will be of most service, namely, in the deepest parts of the lake. Above this impervious covering is found the residual deposit overlying the impervious argillaceous sandstone from which it was derived. The characteristics of this surface layer are its more or less claylike nature at the extreme surface, and its sandy and less cohesive structure as it approaches the solid rock. The nature of this argillaceous sandstone is to crumble and break down to a fine clay where mostly exposed to the weather, the

particles becoming larger and more firm with distance from the surface. The result of this quality is that there is an extensive layer of more or less open structure overlying the impervious rock, capable of storing large quantities of water, and well protected from surface evaporation. From this layer comes the well-sustained flow that appears in many streams during periods when there is little or no rain. This formation also gives additional storage capacity to the lake, so that demands on it will be honored in excess of visible storage. Such condition has been found in the case of the Rio Grande Reservoir, and will be commented on later. What this storage may be is unknown at present, and can only be determined after the lake is filled. This quantity should not be considered in calculations affecting the water supply, but should be left as an asset to be applied to any losses which may have been overlooked or under-estimated.

Rainfall.

The cause of rainfall is to be found in the operation of certain general principles of physics, but is influenced more or less by local conditions. Rainfall is due to the precipitation of the water vapor in the atmosphere which has been derived primarily from the ocean by the process of evaporation. No matter how far this vapor may be driven by the winds, in the course of time the water returns again to the ocean. Evaporation takes place from the surface of every body of water, from the ground, and from vegetation. Whenever for any cause the moisture-laden air is cooled, condensation and consequent precipitation take place.

According to Curtis, three different processes are primarily concerned in the production of rain, acting either singly or in combination. These factors are:

1.—Convective currents;
2.—Hills and mountains which cause deflection of atmospheric currents; and
3.—Cyclonic circulation.

The Isthmus of Panama is in that portion of the globe where the influence of convection is very great, but where cyclonic disturbances are almost absent. Currents due to convection of heat are nearly vertical, and moisture evaporated in a region exposed to these con-

ditions will largely be precipitated before being carried far away. Increased evaporation, therefore, will be followed by increased rainfall, and any change that increases or decreases the former will be followed by a corresponding change in the precipitation. Wherever there are mountain ridges the slopes of which obstruct and deflect upward the prevailing winds, there also will be found a well-watered country and perennial streams. So effective is this cause in producing rainfall that it is of frequent occurrence to find luxuriant vegetation on the windward side of a mountain range while the leeward side is dry and parched, and with meager growth. Although not to extremes, such conditions as these are found in the Canal Zone, with its 72 in. of

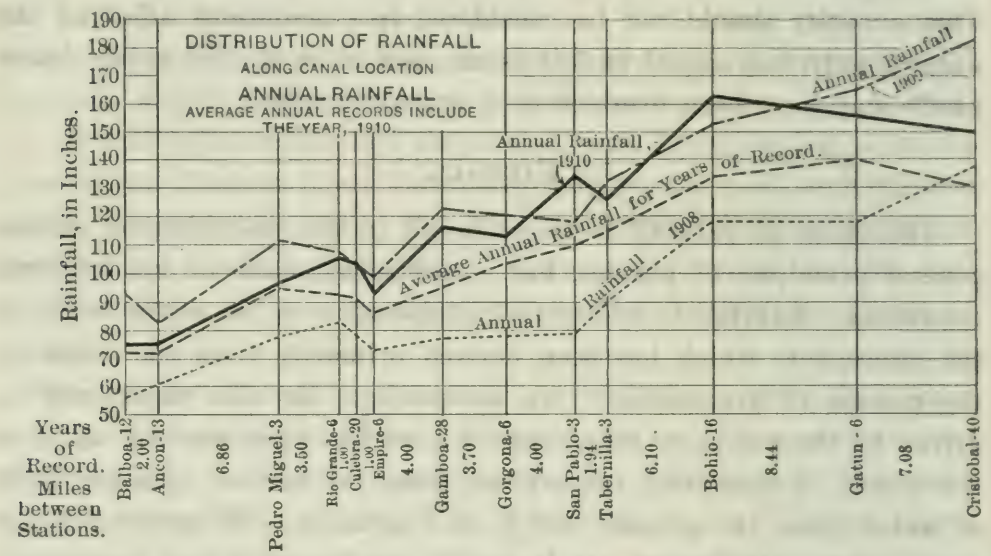


FIG. 14.

precipitation on the Pacific Coast and its 130 in. on the Atlantic, which are averages of long-period records.

The conditions of rainfall following excessive evaporation in this region are best shown in the basin above Alhajuela. There, on the high mountains, the moisture-laden winds from the north and west are forced to drop their loads. Along the Canal Zone and to the southwest, the ridges are of much less elevation, and therefore do not so much disturb the direction of the wind. The results of these conditions are found in the run-off at Alhajuela, with its well-sustained flow, and at Trinidad and Caño, with the opposite conditions.

The distribution of rainfall across the Isthmus is shown by Fig. 14. Culebra, with its annual average of 91 in., is at the limit of the wind-

rd slope, and its rainfall, averaged with that of Cristobal and Gatun, ves a rough approximation of the rainfall conditions north of the ide and over the area in which will lie Gatun Lake and its drainage ea. On the southerly or leeward side of the ridge the rainfall aver- es about 80 in. annually, or less than three-fourths of that on the ndward side. Due to other local conditions, however, the actual nfall on the Gatun Basin is larger than these mean values and re nearly approaches a ratio of 3 to 2.

Records of rainfall have been kept on the Isthmus of Panama in the inity of the Canal Zone for long periods at several of the stations. ble 6 gives the stations which have been in operation for 6 years more, the length of record, and the probable accuracy of the record a basis for prediction of the average conditions which may be pected in the future.

TABLE 6.—Rainfall Stations in Canal Zone.

Station.	Length of record, in years.	Probable error. Percentage.	Station.	Length of record, in years.	Probable error. Percentage
stobal.....	40	1.75	Panama (Ancon).	13	7.00
nboa.....	28	2.25	Empire.....	6	14.00
ebra.....	20	3.25	Gatun.....	6	14.00
io.....	16	4.50	Gorgona.....	6	14.00
ajuela	13	7.00	Rio Grande.....	6	14.00

The figures in Table 6 for probable deviation from the actual erage agree substantially with those given by Mr. A. R. Binnie, and oted by Mr. Rafter.*

In his report on the "Influence of Forests on Stream Flow," Lieut.- l. Edward Burr, M. Am. Soc. C. E., Corps of Engineers, U. S. A., tes as his deduction that 60 years or more of climatic records are cessary from which to draw satisfactory conclusions, and that those awn from records of 40 years or less will probably be misleading or correct. Col. Burr's conclusions regarding length of records neces- y for basing estimates are therefore substantially in accord with similar deductions of Messrs. Binnie and Henry, mentioned pre- usly. From an examination of the curve, Fig. 15, it appears, how- er, that the probable variation from normal, in records of from 35 40 years, is of such small amount that errors due to observation and

* Water Supply and Irrigation Paper No. 85, p. 18.

TABLE 7.—RAINFALL ON THE
Average Quantities for

Station.	Station Average.	January.		February.		March.		April.		May.	
	No. of years.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.
Ancon	13	1.12	1.12	0.76	1.88	0.89	2.77	2.47	5.24	8.66	13.90
Balboa	12	1.28	1.28	0.36	1.64	0.99	2.63	3.92	6.55	6.94	13.49
Miraflores	2	3.88	3.88	1.92	5.80	1.80	7.60	3.72	11.32	9.57	20.89
Pedro Miguel	3	1.64	1.64	1.25	2.89	0.72	3.61	2.88	6.49	12.16	18.65
Rio Grande	6	1.79	1.79	0.69	2.48	0.42	2.90	3.05	5.95	10.54	16.49
Culebra	20	1.92	1.92	0.56	2.48	0.75	3.23	3.78	7.01	11.11	18.12
Camacho	4	1.77	1.77	0.89	2.66	0.82	3.48	2.92	6.40	11.00	17.40
Empire	6	1.02	1.02	0.59	1.61	0.57	2.18	3.65	5.83	9.52	15.35
Gamboa	28	1.98	1.98	0.85	2.83	0.88	3.71	3.62	7.33	10.89	18.22
Alhajuela	11	1.44	1.44	0.69	2.13	0.78	2.91	3.34	6.25	12.35	18.60
El Vigia	2	2.30	2.30	3.64	5.94	1.64	7.58	2.32	9.90	13.00	22.90
Gorgona	6	2.37	2.37	0.76	3.13	1.30	4.43	3.18	7.61	12.20	19.81
San Pablo	3	2.37	2.37	1.57	3.94	1.81	5.75	5.14	10.89	10.09	20.98
Tabernilla	3	2.59	2.59	1.80	4.39	2.39	6.78	4.99	11.77	11.34	23.11
Bohio	16	6.38	6.38	2.13	8.51	2.08	10.59	5.84	16.43	13.93	30.36
Trinidad	3	4.86	4.86	3.70	8.56	4.41	12.97	6.55	19.52	13.47	32.99
Monte Lirio	3	5.01	5.01	5.33	10.34	5.71	16.05	5.94	21.99	14.56	36.55
Gatun	6	4.75	4.75	2.44	7.19	3.49	10.68	3.64	14.32	15.39	29.71
Brazos Brook	4	4.87	4.87	2.38	7.25	4.40	11.65	3.56	15.21	11.92	27.13
Cristobal	40	4.13	4.13	1.45	5.58	1.70	7.28	4.19	11.47	12.31	23.78
Porto Bello	3	12.57	12.57	4.74	17.31	3.60	20.91	8.56	29.47	13.34	42.81
Nombre de Dios	2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

the application of the data from a few stations to a large area may somewhat offset the accuracy presumed for longer records.

Fig. 16 shows graphically the annual rainfall for five stations having the longest records, and Fig. 19 shows the comparative monthly distribution for several stations, including those recorded on Fig. 16. Table 7 gives the mean monthly and accumulated quantities for each station.

Of the 31 rainfall stations now operated, 17 are supplied with rain gauges of the U. S. Weather Bureau standard pattern, 11 have automatic gauges of the tipping-bucket type, and 3 have 14-day automatic gauges of Dutch make. The latter are located in isolated positions, where daily attendance is impracticable.

The Weather Bureau standard rain gauge consists of a hollow metal cylinder, 8 in. in diameter and 20 in. high, made in two sections, a receiver for catching the rainfall and an 8-in. overflow tank for storing any water that may accumulate in excess of the quantity required to fill the measuring tube. The receiver leads by a funnel to the copper measuring tube, approximately 2½ in. in diameter, placed inside the

STHMUS OF PANAMA, IN INCHES.

he Years of Record.

JUNE.		JULY.		AUGUST.		SEPTEMBER.		OCTOBER.		NOVEMBER.		DECEMBER.	
Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.
8.69	22.59	8.18	30.77	7.64	38.41	7.49	45.90	10.43	56.33	10.94	67.27	4.40	71.67
8.36	21.85	10.11	31.96	7.54	39.50	6.36	45.86	9.12	54.98	9.63	64.61	6.45	71.06
16.05	36.94	10.79	47.73	9.22	56.95	10.70	67.55	13.26	80.91	13.34	94.25	10.88	105.13
12.11	30.76	9.22	39.98	9.82	49.80	8.49	58.29	13.10	71.39	13.02	84.41	10.39	94.80
10.68	27.17	12.03	39.20	10.22	49.42	11.29	60.71	13.11	73.82	11.39	85.21	7.02	92.23
9.17	27.29	9.68	36.97	10.57	47.54	11.35	58.89	11.35	70.24	12.52	82.76	8.19	90.95
12.07	29.47	11.93	41.40	10.25	51.65	11.33	62.98	13.71	76.69	14.70	91.39	8.34	99.73
8.57	23.92	10.24	34.16	10.14	44.30	7.93	52.23	14.16	66.39	11.49	77.88	6.61	84.49
9.72	27.94	10.42	38.36	12.20	50.56	10.61	61.17	12.77	73.94	12.54	86.48	7.31	93.79
13.13	31.73	13.85	45.58	13.29	58.87	12.00	78.87	13.53	84.40	14.70	99.10	8.04	107.14
14.48	37.38	14.63	52.01	13.12	65.13	15.39	80.52	17.21	97.73	20.64	118.37	10.20	128.57
9.02	28.83	11.99	40.82	12.37	53.19	13.17	66.36	13.10	79.46	14.37	93.83	8.41	102.24
10.69	31.67	12.21	43.86	10.94	54.80	12.72	67.62	16.14	83.66	15.14	98.80	9.88	108.68
10.57	33.68	10.43	44.11	10.80	54.91	14.41	69.32	18.19	87.51	16.38	103.89	9.85	113.74
12.46	42.82	13.37	56.19	15.34	71.53	14.02	85.55	16.80	102.35	19.39	121.74	11.25	132.99
11.61	44.60	10.72	55.32	13.51	68.83	14.21	83.04	14.49	97.53	23.47	121.00	15.79	136.79
14.38	50.93	16.16	67.09	12.65	79.74	13.32	93.06	14.46	107.52	28.63	136.15	14.74	150.89
13.38	43.09	13.68	56.77	16.10	72.87	10.91	83.78	16.91	100.69	24.03	124.72	15.83	140.55
15.03	42.16	17.45	59.61	15.24	74.85	12.31	87.16	14.76	101.92	26.43	128.35	17.56	145.91
13.23	37.01	16.50	53.51	15.22	68.73	12.56	81.29	14.15	95.44	22.01	117.45	12.58	130.03
16.65	59.46	19.70	79.16	17.17	96.33	11.91	108.24	8.51	116.75	31.85	148.60	30.07	178.67
.....	.....	17.02	.....	10.89	.....	7.31	.....	11.89	.....	20.30	.....	22.43	.....

-in. cylinder. When the measuring tube has become filled, the water s allowed to overflow into the larger cylinder, whence it is drawn off ater and measured. The two cylinders are proportioned so that the neasuring tube has exactly one-tenth of the cross-section area of the eceiving cylinder, therefore $\frac{1}{10}$ in. in depth in the measuring tube epresents 0.01 in. of rainfall. The quantity of rainfall is obtained y measuring the depth in the smaller cylinder with a graduated stick, urning back into the smaller cylinder any rain which may have verflowed into the larger one.

The tipping-bucket gauge is of more complicated design. It is omposed of a metal cylinder, 12 in. in diameter, open at the top, and bout 30 in. high over all. Inside the cylinder, a brass bucket is alanced, having two compartments of the same size and shape. The ucket is adjusted to tip when a quantity of water equal to $\frac{1}{100}$ in. ver the area of the receiving cylinder has accumulated, and spill the contents into the cylinder, where it can be measured with a stick s a check. The bucket is connected electrically with a register on a evolving drum on which is wrapped a sheet of paper graduated in

hour- and five-minute periods. Every tip of the bucket is registered on the paper by a pen or pencil, and as each mark represents $\frac{1}{100}$ in. of rainfall, an automatic record is obtained of the time the storm begins and ends, and the number of marks gives the total quantity and the rate of downpour in a given time.

The Dutch gauges are of somewhat similar construction, except that there is no electrical connection, the entire gauge and recording devices being contained in the same case. On account of direct measurements, the standard 8-in. rain gauge is considered the most reliable indicator of the quantity of precipitation. In order to obtain

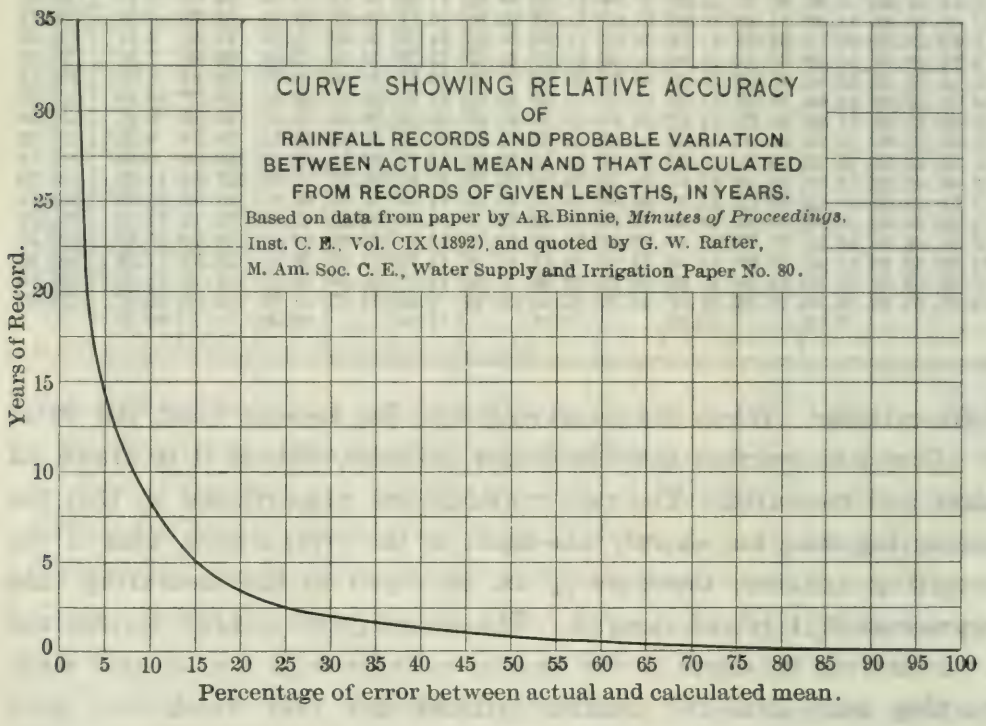

CURVE SHOWING RELATIVE ACCURACY
OF
RAINFALL RECORDS AND PROBABLE VARIATION
BETWEEN ACTUAL MEAN AND THAT CALCULATED
FROM RECORDS OF GIVEN LENGTHS, IN YEARS.
Based on data from paper by A. R. Binnie, Minutes of Proceedings, Inst. C. E., Vol. CIX (1892), and quoted by G. W. Rafter, M. Am. Soc. C. E., Water Supply and Irrigation Paper No. 80.
Years of Record.
0
5
10
15
20
25
30
35
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100
Percentage of error between actual and calculated mean.

FIG. 15.

some idea of the reliability of the automatic gauge, a standard gauge (8-in. diameter) and an automatic gauge (12-in. diameter), were located side by side and daily readings were taken during August and September, 1910. During August, 95% of the measurements in the two gauges agreed exactly, and the remaining 5% were only slightly at variance with each other. The total monthly rainfall by stick measurement was the same in both gauges. During September, 93% of the measurements agreed exactly, while the total monthly rainfall by the standard gauge equalled 101% of the total by the tipping-bucket

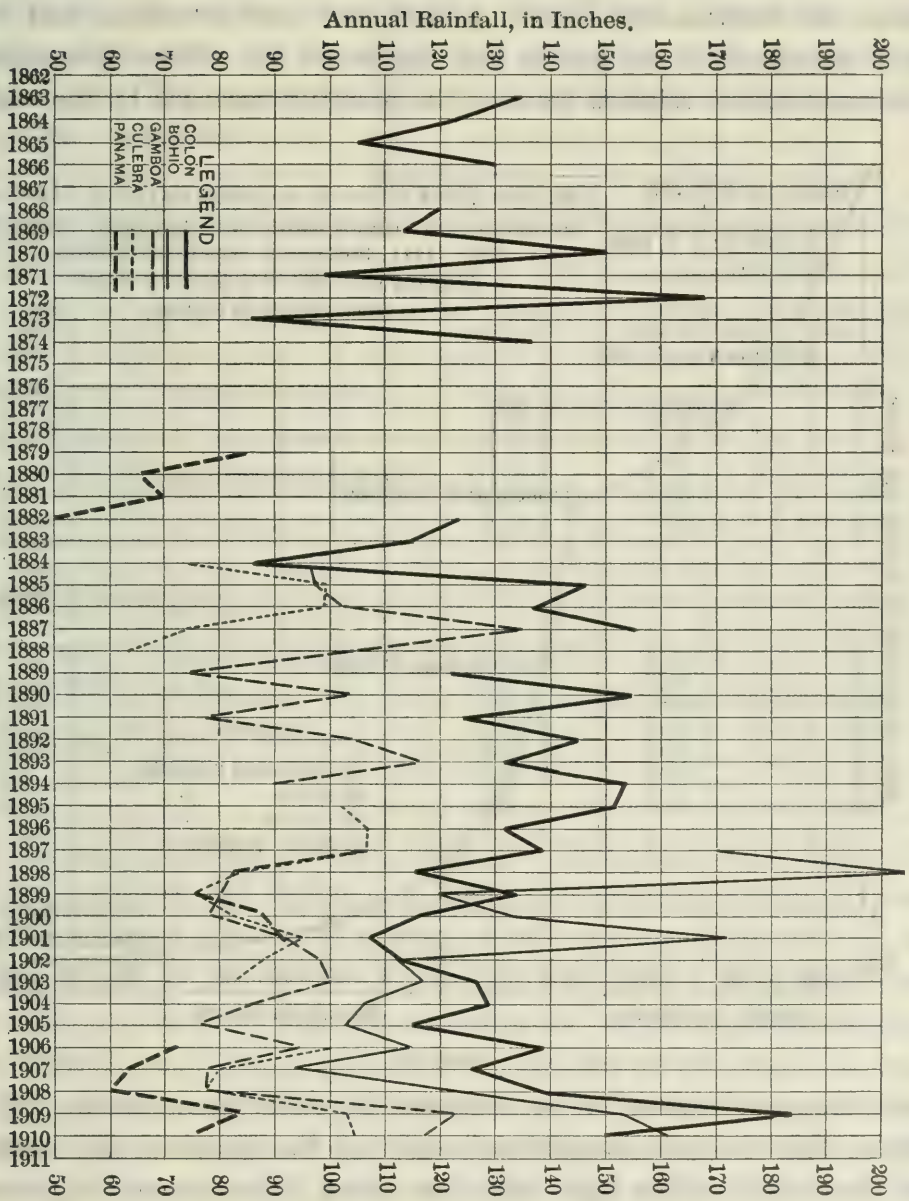

Annual Rainfall, in Inches.
50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
1862 1863 1864 1865 1866 1867 1868 1869 1870 1871 1872 1873 1874 1875 1876 1877 1878 1879 1880 1881 1882 1883 1884 1885 1886 1887 1888 1889 1890 1891 1892 1893 1894 1895 1896 1897 1898 1899 1900 1901 1902 1903 1904 1905 1906 1907 1908 1909 1910 1911
LEGEND
COLON
BOHIO
GAMBOA
CULEBRA
PANAMA

FIG. 16.

gauge. It appears that the records from these two gauges agreed about as closely as would the records of two gauges of the same type if exposed side by side.

Like all automatic devices, however, the records are only reliable when the instruments receive proper care and attention, and when their peculiarities are known and accounted for. Some investigation was made of the relation between the quantity recorded by the register

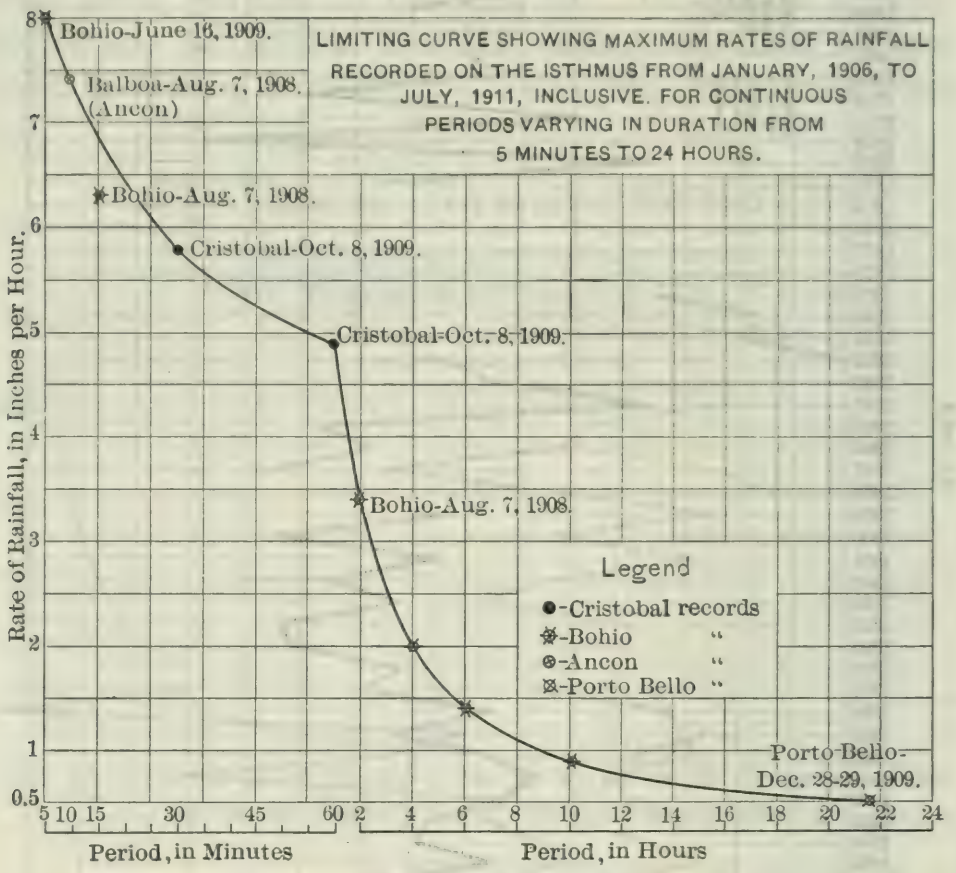


FIG. 17.

and the actual collection as indicated by stick measurement. It was found that the ratio depended largely on the quantity and character of the rainfall. For light and slow rains, the automatic registration accorded closely with the stick measurement, but for heavy rains, a marked deficiency of record appeared with the automatic registers. This loss was due to an appreciable quantity of water which pours into the tipped bucket while it is tipping. Each bucket, therefore, is over-filled, and, as only $\frac{1}{100}$ in. is recorded for each tip, all the water

flowing down the funnel is not measured. The deficiency in the automatic record was found to vary from less than 1% to as much as 8 or 10%, in exceptional cases. There are other factors, also, which contribute to defective registration, among which are lack of care in leveling up the gauge when setting, insufficient lubrication of the moving parts, weak battery power, and poor contacts in the electrical connections. The records in Table 8 are given as an indication of the foregoing conditions.

TABLE 8.

BEST CONDITIONS.

Date.	Gauge.	Gauge record.	Stick measurement.	Percentage registered.
April 25th, 1911	Culebra automatic	2.77 in.	2.84 in.	98
May 3d, 1911	" "	1.18 "	1.19 "	99
May 9th, 1911	" "	1.88 "	1.93 "	97
May 11th, 1911	" "	2.21 "	2.24 "	99
April 25th	Rio Grande automatic	3.12 "	3.12 "	100
May 3d	" " "	1.08 "	1.08 "	100
May 9th	" " "	2.06 "	2.08 "	99
May 11th	" " "	2.92 "	2.97 "	98

WORST CONDITIONS.

April 4th, 1911	Balboa automatic	1.23 in.	1.28 in.	96
April 24th, 25th	" "	2.56 "	2.65 "	97
April 30th	" "	1.56 "	1.75 "	89
May 11th	" "	1.49 "	1.60 "	93
April 24th, 1911	Empire automatic	2.02 "	2.28 "	89
May 3d	" "	0.47 "	0.54 "	89
May 9th	" "	1.86 "	2.07 "	90
May 11th	" "	2.28 "	2.57 "	89

The trouble in the Balboa and Empire gauges was investigated and removed. In the latter case it was due to improper adjustment of the ratchet wheel; in the former it was due to defective leveling. These errors are not the fault of the instrument, but carelessness, inattention, or ignorance would fail to detect them, and the records would be misleading. On this account the total quantity of rain caught by the gauge is always measured by stick, and this reading is used in the records. The automatic record is used only to find the time the shower began and ended, and determine the rates of downpour during various periods.

As it was desirable to get rainfall records from isolated locations, where daily attention was not practicable and where the expense of a long-period gauge was not warranted, some investigation was made

as to the loss by evaporation from a standard rain gauge if the readings were taken at intervals of 2 weeks or longer. During November, 1910, the rainfall was allowed to accumulate in the standard rain gauge at Culebra, measurements being made on the 15th and 30th of the month. The total monthly rainfall by standard gauge showed a quantity slightly in excess of the total by the tipping-bucket gauge, which was close by, the collections in which were measured daily by stick before dumping. In other words, the nightly accumulation from dew and fog in the standard gauge more than equalled the evaporation during the day. It seems, therefore, that semi-monthly measurements of the accumulation in a standard rain gauge during the rainy season will give the total monthly rainfall with a fair degree of accuracy. On account of the construction of the standard rain gauge, its contents are well insulated from the effects of solar radiation until the inner tube is filled, the capacity of the latter being equivalent to 2 in. of rainfall, and, therefore, it seems that fairly reliable results may be obtained during the dry season by visiting the gauges only if necessary two or three times a month.

In regard to the collection from dew and fog, an appreciable quantity of moisture is gathered by night over the receiving surface of a rain gauge. This, however, enters only occasionally into monthly tables, because, if there has been no rain during the night, the moisture which has gathered in the automatic register is thrown out by the observer in the morning unmeasured. The only time this quantity does appear is when rain has fallen, and then it is impossible to separate the precipitation from the two sources. The nightly accumulations from fog and dew are greatest during foggy and clear nights and least during cloudy nights. The quantities range from zero to as much as 0.01 in. for a night. The average for the rainy season is estimated to be about 0.005 in. per night, or 0.15 in. per month. This, of course, does not mean that there is an equivalent of 0.15 in. of rainfall over the whole land surface from fog and dew; it merely means that the thin metal surface of the rain gauge has cooled rapidly by outward radiation, that the warmer air coming in contact therewith has been suddenly cooled, and the vapor condensed and precipitated on the surface of the gauge.

In order to get information concerning rainfall conditions at the heads of streams, three 14-day automatic rain gauges, manufactured in

Holland, were purchased. These gauges have now been in operation more than one year, and the following notes are submitted regarding their value in securing automatic rainfall records at interior points which cannot be visited except at infrequent intervals.

The clocks which actuate the drums on these registers were designed to run 14 days without rewinding, but experience has shown that they cannot be depended on in this climate for periods of more than 10 days. Thus it has been found necessary to visit these gauges at least three times a month.

The drums are driven at such a slow speed that during heavy rains, so frequent in tropical countries such as the Canal Zone, the bucket tips so fast that the record is blurred and the individual tips cannot be counted. The blurred and indistinct character of the record makes it always difficult and at times impossible to determine accurately the quantity of rain falling during heavy showers.

Certain parts of the registering apparatus have been found to get out of order readily, and various parts deteriorate rapidly from rust and other causes when exposed to the climatic conditions obtaining on the Isthmus. When parts of these gauges become worn or broken, it is very difficult, if not impossible, to secure new parts to replace them, as neither the gauge nor any of its parts are manufactured in the United States. If, however, a standard gauge is placed by the side of the 14-day automatic gauge, as a check, it is possible to get daily automatic records of dependable accuracy.

Long-continued rain storms extending over a large area are of infrequent occurrence in the Canal Zone. The rainfall is usually in the form of showers of limited extent, and it has been found that the topography and the location of the rain gauge are most important factors in getting correct results. To investigate this matter somewhat, records from a set of four gauges located in Empire were compared. These gauges are placed almost in a straight line northeast and southwest, and across the valleys of the Obispo and Camacho Rivers from hill to hill. No. 1 is on the east side of the Canal not far from the Empire suspension bridge, at about Elevation 190. No. 2 is the permanent gauge from which the official records are taken, and is about 1 000 ft. southwest of No. 1, and on top of a hill, Elevation 330, on the west side of the Canal, not far from the Division Office. No. 3 is about 2 600 ft. southwest of No. 2, and is in the corral yard, Elevation 220,

a most excellent exposure. No. 4 is at Camacho Reservoir, Elevation 370, and is about 4 600 ft. southwest of No. 3. Gauges Nos. 1 and 3 are in the valleys, while Nos. 2 and 4 are on the hills, but the extreme difference in elevation is not more than 200 ft. Table 9 shows the records, in inches per month, obtained from these gauges:

TABLE 9.—Records of Rain Gauges at Empire.

Month.	No. 1.	No. 2.	No. 3.	No. 4.
June	7.98	5.98	6.43	4.62
July	6.44	4.00	5.94	6.62
August	6.96	5.98	8.50	7.98
September	5.97	5.46	6.20	6.97
October	15.37	14.97	18.05	18.81

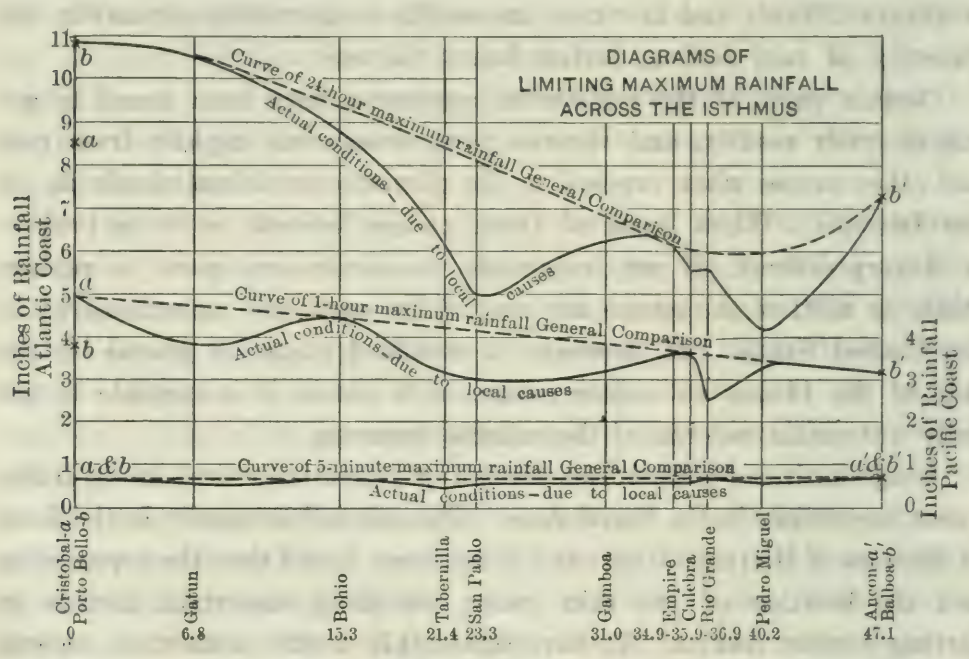


Fig. 18.

From inspection, it seems possible that the annual rainfall hitherto recorded at Empire may be from 12 to 15% too low. Table 10 gives information of a similar character, but mostly illustrating the limited extent of some of these tropical downpours. The records were obtained from Gatun and stations in the vicinity during a heavy rain on May 23d, the distances given being in an air line from Gatun Evaporation Station No. 2. (Gatun Evaporation Stations Nos. 1, 2, and 3 are located on the surface of the water in Gatun Lake.)

TABLE 10.

Station.	Rainfall, in inches.	Air line distance, in miles.	Direction.
Gatun Evaporation Station No. 2.	4.05	0.00	Origin.
" " " " 3.	3.36	0.65	S. E.
" " " " 1.	1.70	1.20	N. W.
Gatun Spillway	0.89	1.70	N. W.
Monte Lirio	3.75	2.90	N. E.
Brazos Brook	1.90	5.00	N. E.
Bohio	3.75	5.90	S. E.
Cristobal	2.42	6.90	North.

The diagram, Fig. 19, shows the frequency of heavy downpour on both sides of the Isthmus. It will be seen that the heaviest rains occur on the Atlantic side, and that there, too, are to be found the least number of days without any rain at all. This diagram represents average conditions.

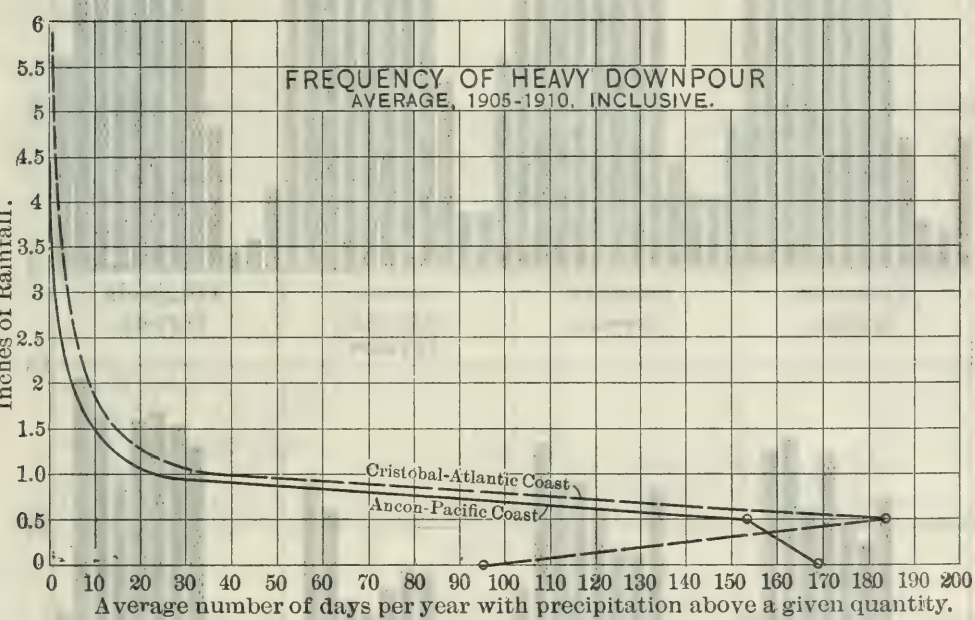


FIG. 19.

Storms giving a greater precipitation than 6.00 in. on the Atlantic or 4.50 in. on the Pacific side in 24 hours are of infrequent occurrence. During the past five years (June, 1906, to July, 1911), there have been 78 storms in which the precipitation has exceeded 4.00 in. Of these, 41 have had a precipitation of not more than 5.00 in., and 70 not more than 7.50 in. The rainfall in each of the remaining storms was: one not more than 8.00 in., one not more than 8.50 in., four not more

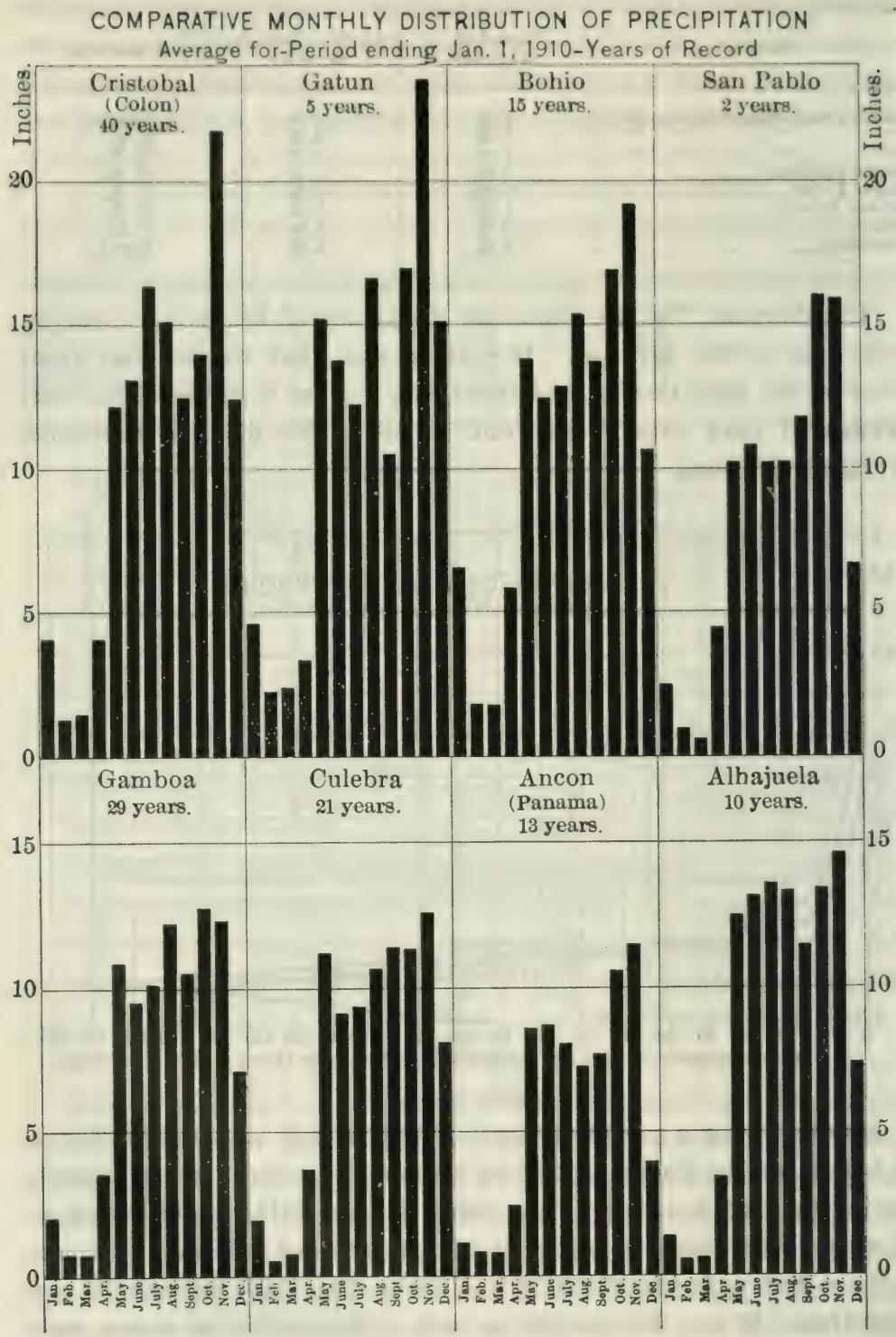

COMPARATIVE MONTHLY DISTRIBUTION OF PRECIPITATION
Average for-Period ending Jan. 1, 1910–Years of Record
Inches.
Cristobal
(Colon)
40 years.
Gatun
5 years.
Bohio
15 years.
San Pablo
2 years.
Gamboa
29 years.
Culebra
21 years.
Ancon
(Panama)
13 years.
Alhajuela
10 years.
0
5
10
15
20
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.

FIG. 20.

than 9.00 in., and two not more than 11.00 in. During this period the rainfall on the Pacific side has exceeded 4.50 in. at one or more stations on six different dates, or an average of a little more than once each year. On the Atlantic side the rainfall has exceeded 6.00 in. at one or more stations on fifteen different dates, or an average of about three times per year. It appears, therefore, that a rainfall of 6 in. in 24 hours is of not infrequent occurrence during the year, and that twice in 5 years a downpour of from 10.50 to 11 in. may be expected. Fig. 17 is a graphical representation of the extreme maximum limits of rainfall during the years of record.

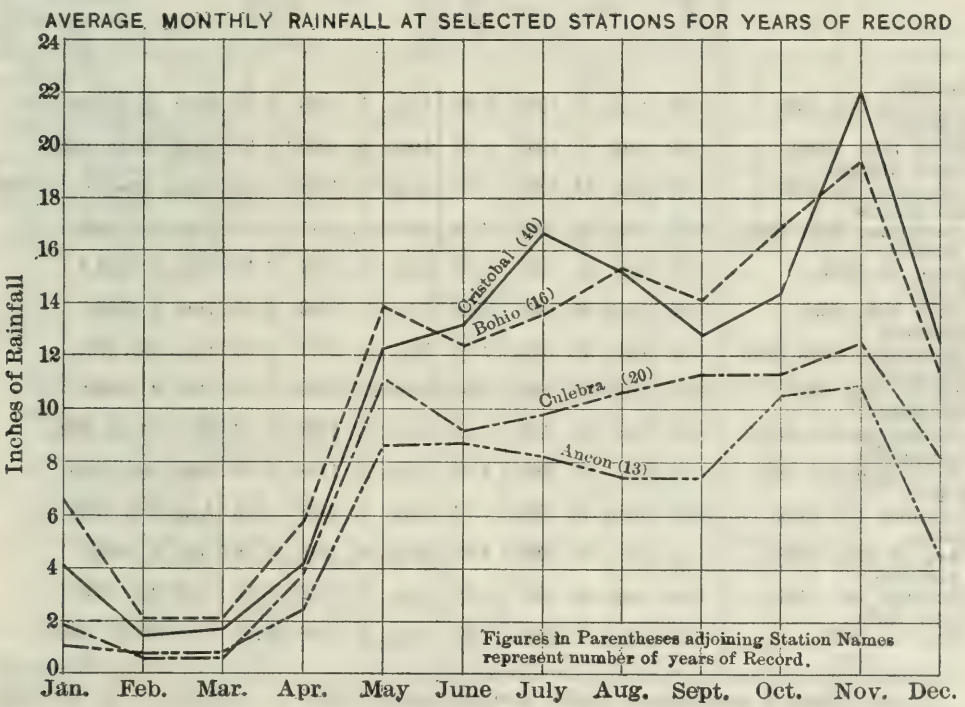


FIG. 21.

Fig. 21 shows the average monthly rainfall at four stations having long-time records. These stations indicate such conditions as are found from ocean to ocean across the Isthmus, and show that even in the so-called dry season there is an appreciable quantity of rainfall. The diagrams, Figs. 22 and 23, show, respectively, the maximum and minimum monthly precipitation for the years of record for the stations mentioned above. The greatest quantity recorded in the Zone was at Colon in November, 1862. Quantities exceeding this, however, were recorded at Porto Bello in 1909, where 45.03 in. fell in November and

58.17 in. in December. The total annual rainfall for this station in 1909 was 237.28 in., which is the maximum of record on the Isthmus.

TABLE 11.—MAXIMUM RAINFALL IN THE CANAL ZONE, OCTOBER 1ST, 1905, TO JUNE 30TH, 1911.

Stations.	Maximum Rainfall.					
	5 Minutes.		1 Hour.		24 Hours.†	
	Inches.	Date.	Inches.	Date.	Inches.	Date.
Ancon, (October 1st, 1905)....	0.64	Aug. 7, 1908.	2.89	Aug. 7, 1908.	6.37	Nov. 16–17, 1906.
Balboa, (June 10th, 1906).....	0.63	Aug. 7, 1908.	5.86	June 2, 1906.	7.57	Nov. 16–17, 1906.
Pedro Miguel, (January 1st, 1908)....	0.60	Nov. 11, 1908.	3.30	Aug. 27, 1908.	4.56	Sept. 30–Oct. 1, 1909.
Rio Grande, (December 29th, 1905).	0.75	July 24, 1908.	2.74	Apr. 25, 1911.	6.00	Dec. 2–3, 1906.
Culebra, (July 1st, 1906).....	0.64	May 2, 1908.	3.69	Oct. 16, 1907.	5.55	Dec. 3, 1906.*
Empire, (July 18th, 1906).....	0.60	July 25, 1906.	3.63	Oct. 1, 1909.	6.15	Dec. 3, 1906.*
Gamboa, (November 18th, 1905).	0.59	July 27, 1908.	3.32	May 11, 1911.	6.56	Dec. 2–3, 1906.
Alhajuela, (March 31st, 1907).....	0.60	July 20, 1909.	3.40	Dec. 28, 1909.	8.19	Dec. 3, 1906.*
San Pablo, (November 1st, 1907)..	0.60	Oct. 29, 1908.	3.10	Oct. 29, 1908.	5.14	Nov. 11–12, 1909.
Tabernilla, (November 1st, 1907)..	0.50	Oct. 31, 1909.	3.09	Aug. 18, 1908.	6.22	Sept. 4–5, 1910.
Bohio, (October 1st, 1905)....	0.67	June 16, 1909.	4.51	Aug. 7, 1908.	8.85	Aug. 7–8, 1908.
Gatun, (August 24th, 1907)....	0.61	July 16, 1908.	3.82	May 26, 1910.	10.48	Dec. 3, 1906.*
Cristobal, (October 1st, 1905)....	0.64	Aug. 25, 1909	4.90	Oct. 8, 1909.	8.53	Dec. 2–3, 1906.
Porto Bello, (May 1st, 1908).....	0.64	Aug. 7, 1908.	3.77	Aug. 7, 1908.	10.86	Dec. 28–29, 1909.

NOTE.—Dates in parentheses under station names refer to installation of automatic rainfall registers.

*No automatic record on these dates. Total for 24 hours ending at noon.

† Maximum rainfall for any 24 consecutive hours.

In Table 11 may be found the record of the maximum rainfalls in the Canal Zone, October 1st, 1905, to June 30th, 1911, for periods of 5 min., 1 hour, and 24 hours. The curves on Fig. 16 show the relative distribution of excessive precipitation with relation to geographical location. Table 12 gives the periods of maximum rainfall for Cristobal, Gamboa, and Bohio.

Table 13 is compiled from the reports of the U. S. Weather Bureau, and is given for the purpose of showing the mean monthly rainfall

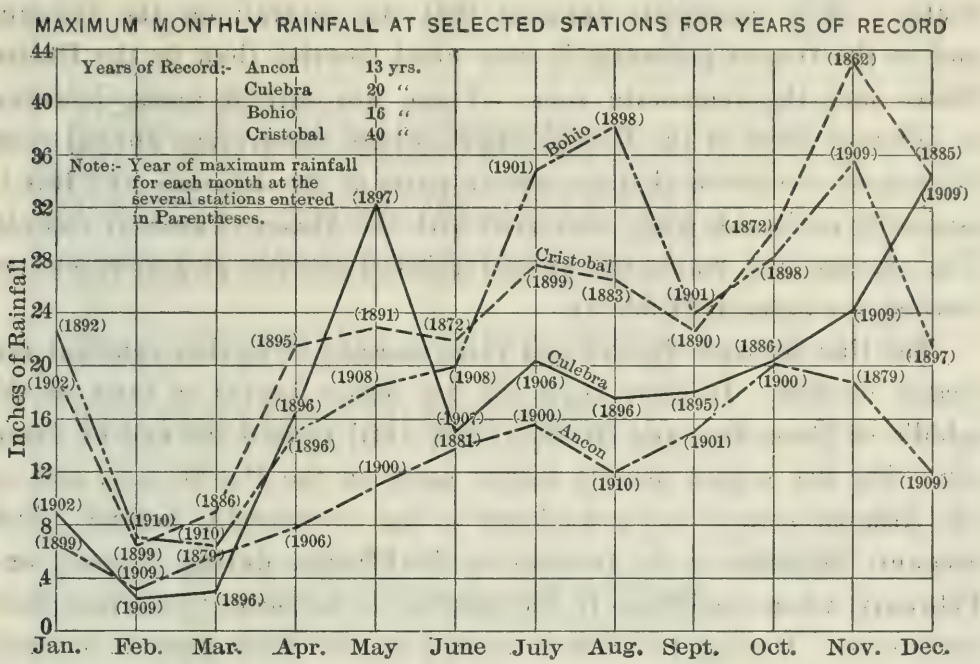

MAXIMUM MONTHLY RAINFALL AT SELECTED STATIONS FOR YEARS OF RECORD
Years of Record:- Ancon 13 yrs.
Culebra 20 "
Bohio 16 "
Cristobal 40 "
Note:- Year of maximum rainfall for each month at the several stations entered in Parentheses.
Inches of Rainfall
Bohio
Cristobal
Culebra
Ancon
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 22.

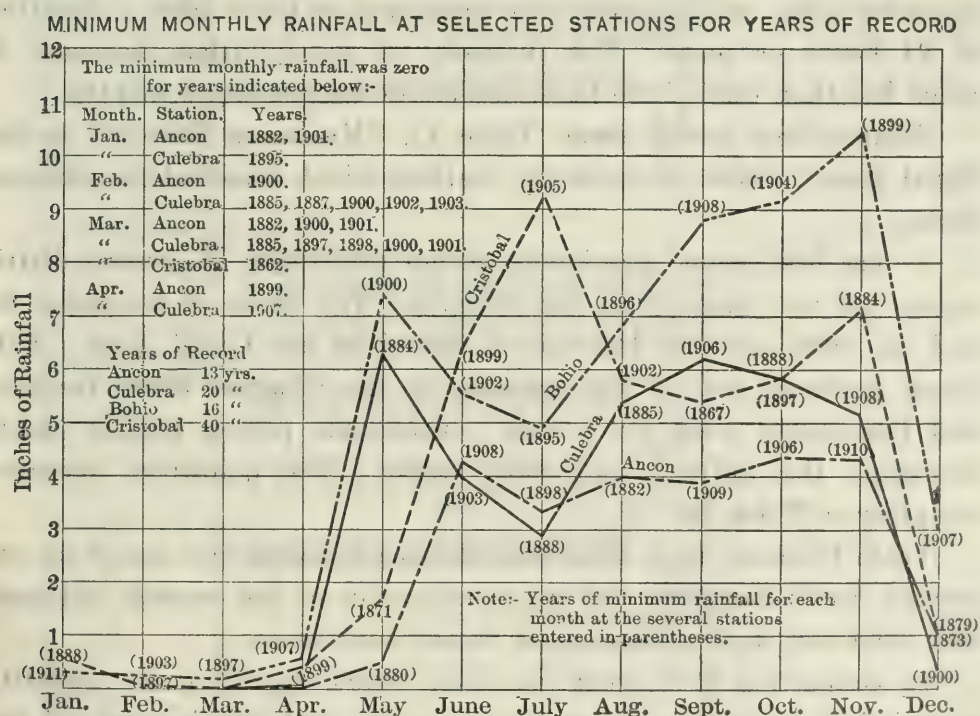

MINIMUM MONTHLY RAINFALL AT SELECTED STATIONS FOR YEARS OF RECORD
The minimum monthly rainfall was zero for years indicated below:-
Month. Station. Years.
Jan. Ancon 1882, 1901.
" Culebra 1895.
Feb. Ancon 1900.
" Culebra 1885, 1887, 1900, 1902, 1903.
Mar. Ancon 1882, 1900, 1901.
" Culebra 1885, 1897, 1898, 1900, 1901.
" Cristobal 1862.
Apr. Ancon 1899.
" Culebra 1907.
Years of Record
Ancon 13 yrs.
Culebra 20 "
Bohio 16 "
Cristobal 40 "
Note:- Years of minimum rainfall for each month at the several stations entered in parentheses.
Inches of Rainfall
Cristobal
Bohio
Culebra
Ancon
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 23.

at selected stations covering geographically the whole of the United States. It is commonly believed that the rainfall on the Isthmus and in the tropics generally is very much heavier than in the United States and the temperate zone. There are several areas, however, in different parts of the United States, where the average annual rainfall equals or exceeds that on certain parts of the Isthmus, and this is especially noticeable when compared with the Ancon (Panama) records. The average and maximum annual rainfall records at a few selected stations are given in Table 14.

The line between the dry and rainy seasons is neither constant nor clearly marked. In some years the dry season begins as early as the middle of December, and in others not until toward the end of January. The dry season usually begins early on the Pacific side, and on the Atlantic side it has a tendency to lag considerably behind. This accounts for some of the freshets in the Chagres during January and February, when conditions in the interior do not seem to warrant their occurrence. During the first 6 months of the rainy season, the rain comes in heavy local showers, and the downpour is usually of short duration. In November and December, rains of a more general character occur, which cover wide areas and at times have a duration of 24 hours or more. The intensity of precipitation, however, is much less than that of the local showers of the preceding months.

Summarizing partly, from Table 11, "Maximum Rainfall in the Canal Zone," Table 15 shows the stations which recorded the extreme limits.

As has been stated previously, severe rainstorms of general distribution are not frequent on the Isthmus. The storm of December 2d and 3d, 1906, was the heaviest of record in the Canal Zone. This storm produced one of the greatest of the Chagres River freshets, and interrupted work for a very considerable period, besides bodily damaging the railroad and other works. The quantities recorded are given in Table 16.

Table 17 shows that, as a rule, the heavy downpours are of an extremely local character, and an examination of the records indicates this condition during showers of recent occurrence.

In connection with what has been stated regarding the quantity of rainfall, it may be interesting to note that the location of the heaviest known rainfall in the world is in the district of Assam,

TABLE 12.—Periods of Minimum Rainfall.

(Consecutive Periods—Calendar Months.)

Length of period.	Cristobal.		Bohio.		Gamboa.	
	Amount, in inches.	Period.	Amount, in inches.	Period.	Amount, in inches.	Period.
1 month..	0.04	February, 1897.....	0.00	February, 1897.....	0.00	(3 months) Feb., 1891, Mch., 1897, Mch., 1898.
2 months.	0.32	February and March, 1897.....	0.25	February and March, 1897.....	0.12	February and March, 1898.
3 " .	2.01	Jan. to Mar., 1885, inclusive...	2.14	Feb. to April, 1905, inclusive..	0.73	Jan. to March, 1908, inclusive.
4 " .	3.35	Jan. to Apr., 1885, " ...	5.13	Jan. " April, 1907, " ..	1.36	Jan. to April, 1907, "
5 " .	6.97	Dec., 1884, to Apr., 1885 " ...	9.57	Dec., 1907, to April, 1908 " ..	2.37	Dec., 1900, to April, 1901 "
6 " .	14.02	Nov., 1884, " Apr., 1885 " ...	20.95	Nov., 1907, " April, 1908 " ..	13.24	Dec., 1900, " May, 1901 "
7 " .	21.93	Nov., 1884, " May, 1885 " ...	31.35	Dec., 1902, " June, 1903 " ..	20.92	Dec., 1900, " June, 1901 "
8 " .	30.59	Oct., 1884, " May, 1885 " ...	42.05	Dec., 1902, " July, 1903 " ..	29.23	Nov., 1897, " June, 1898 "
9 " .	39.96	Sept., 1884, " May, 1885 " ...	53.09	Dec., 1902, " Aug., 1903 " ..	38.73	Dec., 1890, " Aug., 1891 "
10 " .	53.23	Aug., 1884, " May, 1885 " ...	66.16	Nov., 1902, " Aug., 1903 " ..	48.65	Nov., 1890, " Aug., 1891 "
11 " .	68.82	July, 1884, " May, 1885 " ...	77.44	June, 1904, " April, 1905 " ..	59.12	Nov., 1890, " Sept., 1891 "
1 year....	79.14	June, 1884, " May, 1885 " ...	89.68	May, 1904, " April, 1905 " ..	73.55	Dec., 1907, " Nov., 1908 "
2 years...	196.61	June, 1883, " May, 1885 " ...	191.36	June, 1904, " May, 1906 " ..	150.69	May, 1899, " April 1901 "
3 " ...	317.80	June, 1882, " May, 1885 " ...	297.72	June, 1904, " May, 1907 " ..	238.51	May, 1898, " April, 1901 "
4 " ...	422.83	June, 1881, " May, 1885 " ...	396.50	May, 1904, " April, 1908 " ..	320.63	Oct., 1897, " Sept., 1901 "
5 " ...	575.37	May, 1898, " April, 1903 " ...	520.76	June, 1902, " May, 1907 " ..	418.44	Feb., 1904, " Jan., 1909 "
6 " ...	707.37	Aug., 1898, " July, 1904 " ...	621.20	May, 1902, " April, 1908 " ..	513.81	Dec., 1902, " Nov., 1908 "
7 " ...	828.41	Aug., 1898, " July, 1905 " ...	747.81	Feb., 1902, " Jan., 1909 " ..	605.62	Feb., 1902, " Jan., 1909 "
8 " ...	942.91	Aug., 1898, " July, 1906 " ...	934.23	Jan., 1901, " Dec., 1908 " ..	697.28	Feb., 1898, " Jan., 1906 "
9 " ...	1 081.81	July, 1897, " June, 1906 " ...	1 052.76	Apr., 1899, " Mar., 1908 " ..	780.87	Nov., 1897, " Oct., 1906 "
10 " ...	1 209.03	June, 1896, " May, 1906 " ...	1 181.52	Mar., 1899, " Feb., 1909 " ..	861.92	Dec., 1898, " Nov., 1908 "
11 " ...	1 342.15	June, 1896, " May, 1907 " ...	1 383.89	Feb., 1898, " Jan., 1909 " ..	949.07	Jan., 1898, " Dec., 1908 "
12 " ...	1 480.41	May, 1896, " April, 1908 " ...	1 558.70	Jan., 1897, " Dec., 1908 " ..	1 056.40	Jan., 1897, " Dec., 1908 "
13 " ...	1 620.13	Nov., 1895, " Oct., 1908 " ...				
14 " ...	1 776.24	Jan., 1895, " Dec., 1908 " ...	Note:—Length of rainfall records at Bohio, twelve years.		Note:—Gamboa rainfall records for 1895 and 1896 incomplete.	
15 " ...	1 924 13	Jan., 1893, " Dec., 1907 " ...				
16 " ...	2 058 16	Aug., 1892, " July, 1908 " ...				
17 " ...	2 200.87	July, 1891, " June, 1908 " ...				
18 " ...	2 331.84	Jan., 1891, " Dec., 1908 " ...				
19 " ...	2 476.68	Nov., 1890, " Oct., 1909 " ...				
20 " ...	2 607.64	Jan., 1889, " Dec., 1908 " ...				

TABLE 13.—Mean Annual Rainfall, in Inches, by

District.	1		2		3		4		5	
City.	Boston.	Philadelphia.	Norfolk.	Jacksonville.	Knoxville.	Cincinnati.	Cleveland.	Marquette.	Peoria.	St. Paul.
Years of Record.	96	39	38	56	38	74	51	38	57	72
January	3.74	3.23	3.30	2.82	4.78	3.27	2.52	2.08	1.77	0.89
February	3.51	3.35	3.78	3.25	4.75	3.18	2.61	1.73	2.04	0.79
March	4.14	3.43	4.35	3.39	5.29	3.77	2.95	2.06	2.72	1.44
April	3.81	2.92	3.31	2.70	4.47	3.20	2.73	2.13	3.14	2.41
May	3.66	3.30	4.13	3.93	3.74	3.96	3.78	3.20	3.88	3.44
June	3.14	3.27	4.12	5.64	4.27	4.20	3.95	3.57	3.78	4.10
July	3.51	4.14	5.91	6.37	4.07	3.77	3.80	3.06	3.98	3.53
August	4.15	4.69	6.05	6.60	4.03	3.61	3.09	2.74	3.14	3.47
September	3.44	3.36	4.01	8.16	2.86	2.79	3.75	3.61	3.60	3.39
October	3.71	3.01	3.75	4.60	2.54	2.59	2.96	3.08	2.24	1.99
November	4.11	3.11	2.76	2.21	3.51	3.19	3.22	2.72	2.31	1.38
December	3.78	3.07	3.42	2.86	4.04	3.38	2.74	2.44	2.15	0.97
Totals	44.70	40.88	49.29	52.53	48.35	40.91	38.10	32.42	34.75	27.80

British India. The portion of the district affected is comparatively small in area, and is in the foot-hills on the south slope of the Himalaya Mountains. The average annual rainfall there is about 475 in., and the total for individual years has exceeded 800 in. Practically all the

TABLE 14.—Average and Maximum Annual Rainfall at a Few Selected Stations.

Station.	Location.	Annual Rainfall for Years of Record, in Inches.	
		Average.	Maximum.
Cristobal	Canal Zone	130.03	183.41
Clearwater, Wash	Pacific Coast	128.45	151.56
Bowmans Dam, Cal	West Slope, Cascade Mountains	75.63	135.70
Astoria, Ore	Pacific Coast	75.35	108.30
Ancon	Canal Zone	71.67	91.42
Ashford, Wash	West Slope, Cascade Mountains	70.24	84.25
McKenzie Bridge, Ore	West Slope, Cascade Mountains	69.84	86.11
Moline, Fla	North Gulf Coast	66.19	77.27
Mobile, Ala	North Gulf Coast	61.80	91.18
Franklin, La	North Gulf Coast	60.92	79.51
Hatteras, N C	South Atlantic Coast	60.21	102.04

Months, at Selected Stations in the United States.

6			7		8		9		10	11	12	13
Leavenworth.	Williston, N. D.	Helena.	New Orleans.	Fort Sill.	Galveston.	El Paso.	Prescott.	Denver.	Salt Lake City.	San Diego.	San Francisco.	Portland, Ore.
72	29	28	74	39	39	50	42	37	35	60	60	58
1.10	0.60	1.00	4.54	1.13	3.41	0.42	1.56	0.48	1.33	1.76	4.82	6.32
1.37	0.45	0.66	4.28	1.19	3.02	0.47	1.96	0.47	1.48	1.96	3.63	4.96
1.87	0.67	0.83	4.56	1.55	2.90	0.29	1.71	0.96	2.04	1.46	3.32	4.74
2.92	1.23	1.06	4.53	2.84	3.07	0.19	0.82	2.09	2.07	0.61	1.68	2.94
4.49	2.15	2.13	4.06	5.14	3.36	0.28	0.55	2.57	2.16	0.34	0.74	2.38
5.15	3.61	2.26	5.39	3.72	4.19	0.59	0.24	1.46	0.77	0.06	0.02	1.77
4.22	1.96	1.13	6.53	3.14	3.96	1.67	2.85	1.64	0.50	0.06	0.02	0.75
4.21	1.32	0.69	5.65	2.97	4.75	1.88	3.20	1.35	0.85	0.11	0.02	0.58
3.58	0.92	1.13	4.49	3.14	5.72	1.64	1.05	0.90	0.91	0.08	0.31	1.69
2.42	0.32	0.77	3.25	2.58	4.33	0.87	0.78	0.97	1.43	0.34	1.03	3.11
2.09	0.56	0.76	3.81	1.79	3.90	0.57	1.11	0.56	1.39	0.95	2.62	6.00
1.38	0.57	0.79	4.54	1.66	3.70	0.44	1.57	0.60	1.40	1.83	4.64	7.21
34.80	14.86	13.21	55.63	30.85	46.31	9.31	17.40	14.05	16.33	9.56	22.83	42.45

rain falls in the 6 months of the summer monsoon, May to October, inclusive. The maximum daily record for this section is reported as 39 in.

TABLE 15.—Extreme Limits of Rainfall.

Period.	Amount, in inches.	Station.	Date.
5 minutes	0.75	Rio Grande,	July, 1908.
1 hour	5.86	Balboa,	June, 1906.
24 hours	10.86	Porto Bello,	Dec., 1909.
1 month	58.17	" "	" 1909.
1 year	237.28	" "	Year, 1909.

TABLE 16.—Rainfall in Storm of December 2d and 3d, 1906.

Station.	Inches.	Station.	Inches.	Station.	Inches.
Ancon	3.18	Bohio	6.13	Empire	6.15
Rio Grande	5.58	Brazos Brook	8.96	Alhajuela	8.19
Camacho	5.53	Balboa	2.41	Gatun	10.48
Gamboa	6.29	Culebra	5.55	Cristobal	8.47

TABLE 17.—Local Character of Heavy Downpours.

Station.	Rainfall.	Station.	Rainfall.	Distance between stations, in miles.	Date.
Cristobal	2.44	Gatun	0.01	6.4	May 6th, 1911
Gorgona	5.45	San Pablo	1.27	3.5	May 13th, 1911
Gatun *	4.05	Gatun	0.89	1.2	May 23d, 1911
Monte Lirio	6.76	Brazos Brook	0.97	6.0	Feb. 8th, 1910
Gamboa	3.21	Empire	0.52	4.0	July 3d, 1910
Cristobal	2.94	Brazos Brook	0.34	2.5	Oct. 5th, 1910
Rio Grande	3.25	Culebra	1.62	0.8	Nov. 11th, 1910

* Evaporation Station No. 2.

Run-Off.

All the water that flows in a stream comes from the rainfall, but by no means all the rainfall finds its way into the stream. What proportion ultimately does appear depends on many conditions and combinations, the principal factors of which are elevation, topography, geology, and vegetation. The proportions of the rainfall ordinarily found running in northern streams may be seen in Table 18, abstracted from the Report of the Board of Public Works, Philadelphia, Pa.

TABLE 18.—Average Annual Yields of Sundry Water-sheds to October 1st, 1908.

Water-sheds.	Period covered, in years.	Area, in square miles.	Average rainfall, in inches.	Average rainfall flowing off, in inches.	Percentage flowing off.	Average yield, in cubic feet per second per square mile of drainage.	Average yield, in cubic feet per second per square mile of drainage area, for each inch of rainfall.
Perkiomen, at Frederick	25	152.0	47.179	23.276	49.313	1.7104	0.0360
Neshaminy, below Forks	25	139.3	48.113	23.111	48.033	1.6979	0.0350
Tohickon	25	102.2	48.751	27.264	55.920	2.1963	0.0450
Schuylkill	10	1 915.0	48.139	21.564	44.800	1.5842	0.0330
Sudbury, Mass	33	75.2	45.99	22.387	48.680	1.6451	0.0380

It appears that, in streams of the character of those in Table 18, from 45 to 55% of the rainfall is ordinarily collectable, and that the yield may be taken as from 1.5 to 2.2 cu. ft. per sec. per sq. mile of drainage area. In the case of the Isthmian streams, for the past 10 years, the ratio of average annual run-off to rainfall has been more than 60%, and the average annual yield per square mile of the Chagres

River has averaged between 5.75 and 6.06 cu. ft. per sec., with some monthly averages of nearly twice as much.

For the purpose of measuring the discharge of the rivers and streams, gauging stations have been maintained at Gatun, Bohio, Gamboa, and Alhajuela, all on the Chagres River proper. River stage stations have also been maintained at Lagartera, on the Trinidad River, and at Monte Lirio, on the Gatun River, where several gaugings per month are usually taken. Besides this, a hydrographer makes regular trips to the principal secondary streams for the purpose of gauging, the intention being to get measurements at different gauge heights. On account of back-water from the lake, the Trinidad and Gatun River stations were abandoned in April, 1911, and December, 1910, respectively, after records had been obtained for about 4 years at each place. Bohio was abandoned as a gauging station in October, 1910, but is still maintained as a river stage station. Automatic water stage registers are in operation at all the regular stations, and a continuous record of river heights has been obtained since gaugings were begun.

A meter-rating station is maintained at Bohio, and all meters used on the work are rated and adjusted as nearly as possible at regular intervals.

The regular field work is carried on by the hydrographers in charge of the several stations, and after being worked up, the data obtained are sent to the central office for checking and filing. Gaugings are made frequently, and at as many different elevations as possible, in order to check the discharge curves. This course is necessary on account of the continuously changing cross-sections of the river bed, due to the shifting gravel bars. Cable stations are maintained at Gatun, Gamboa, and Alhajuela, and the gauging is done from cars, except in very low water, when wading or a boat is resorted to. All other gaugings are made by wading, under ordinary conditions, and by estimate of quantity passing under bridges, in times of freshet.

At Gatun the entire run-off from the Chagres Basin has passed through the spillway since April, 1910, and gaugings have been made regularly. Up to the dry season of 1911, and before the installation of the cable, gaugings were made from a boat. During periods of extremely low water, it was possible to work in the spillway itself, where, on account of the even flow and the shape of the concrete sec-

tion, very good measurements of the minimum discharge of the stream in 1911 were obtained. At the same time simultaneous gaugings were made at the cable station below, and from comparison with the spillway gaugings, it appeared that the results at the regular station were satisfactory.

Tables 19 to 30, inclusive, give the discharge measurements in various units for the Chagres River at Alhajuela, Gamboa, Bohio, and Gatun. Unless otherwise noted, the quantities are from actual gaugings—floats previous to 1907, and current-meter work subsequently.

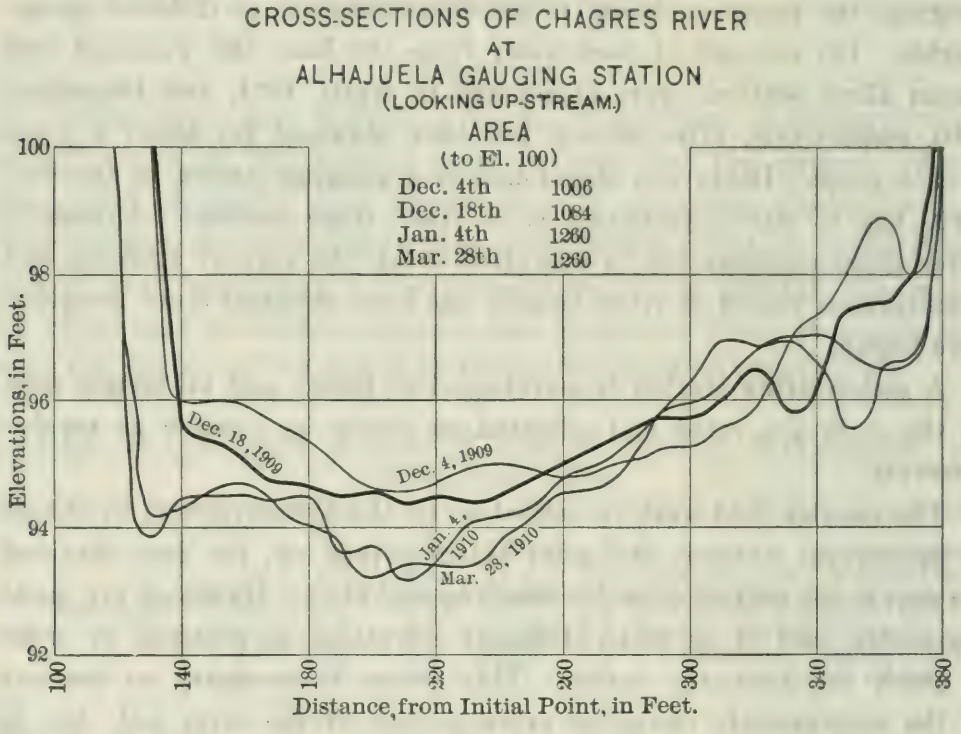


FIG. 24.

The discharge measurements for the stations on the Trinidad and Gatun Rivers for the years of record are given in Tables 31 and 32.

The Chagres River above Bohio has a constantly changing bed, and in order to get reliable measurements and estimates of the flow it is necessary to be constantly on the watch for changes in cross-section. After every freshet there is a change, and oftentimes of a most remarkable character. Some idea of this condition, perhaps, may be found from the fact that from one bar the Panama Railroad dredged some 60 000 cu. yd. of gravel in 1909, 90 000 cu. yd. from the same place in 1910, and about 95 000 cu. yd. in 1911. This gravel is dredged and stored during the dry season to be used as ballast on the new

railroad location. In spite of the removal, the pit is filled again during early flood season by most excellent gravel of various sizes, and, of course, cleanly washed. The cross-sections shown by Figs. 24 to 27, inclusive, were taken at the several stations, as required by the gauging work, and show the variations in area due to freshet scour. This action is perhaps best illustrated by comparing the cross-sections taken on December 18th, 1909, and January 4th, 1910, before and after one of the largest freshets recorded on this river. This rise began at Alhajuela on December 26th, and in the 9 days intervening, up to Jan-

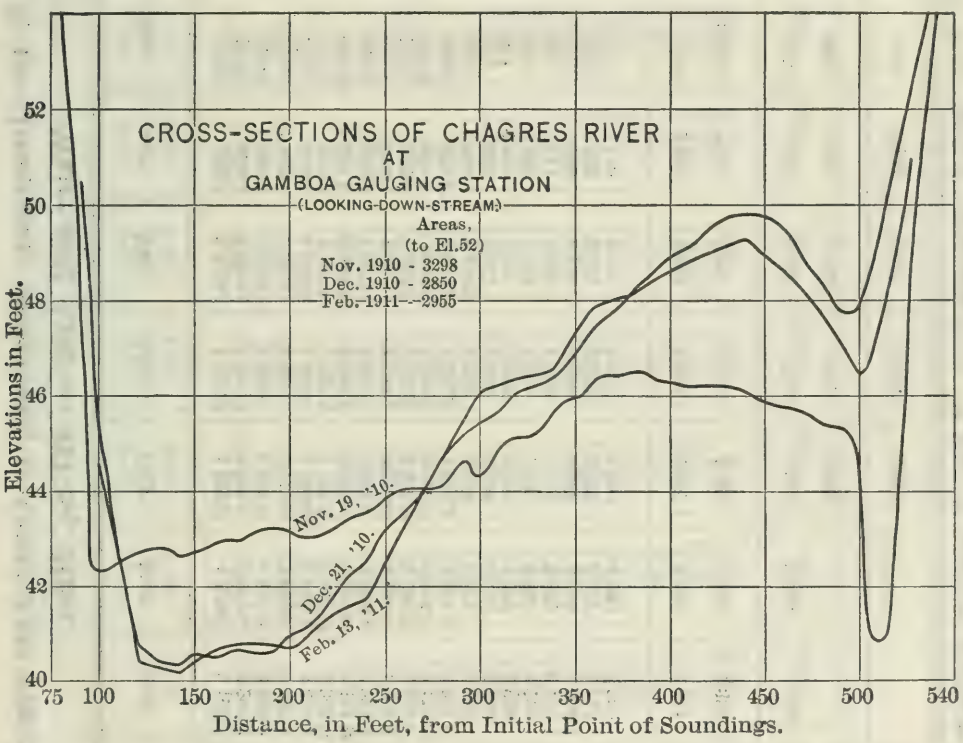


FIG. 25.

uary 4th, when the cross-section was taken, there had been a change in area of 176 sq. ft. below Elevation 100. The Gamboa station was not in operation at this time, but between December 21st, 1910, and February 13th, 1911, there was an increase of 95 sq. ft. in section at this station, due to a smaller freshet which caused high-water conditions for about 12 hours. The relation of change in cross-section is illustrated by the curves of area, velocity, and discharge for Gamboa shown by Fig. 28. These curves are made up from actual gaugings, and the change in cross-section producing the change in discharge can be seen by inspection of Fig. 25.

TABLE 19.—Discharge, Alhajuela, in Cubic Feet per Second.

Water-shed Area = 427 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	2 140*	1 095*G*	918*G*	883*G*	3 249*G*	4 061*G*	4 097*G*	4 309*G*	5 191*G*	4 909*G*	3 002*G*	4 485*G*	3 195
1891	1 554*G*	777*G*	494*G*	424*G*	2 649*G*	1 801*G*	2 861*G*	2 119*G*	2 296*G*	3 390*G*	5 686*G*	3 885*G*	2 328
1892	1 165*G*	918*G*	812*G*	1 695*G*	4 203*G*	2 649*G*	5 651*G*	4 626*G*	3 320*G*	4 521*G*	5 509*G*	4 627*G*	3 308
1893	1 731*G*	1 660*G*	1 024*G*	2 402*G*	2 331*G*	2 437*G*	2 719*G*	3 602*G*	2 967*G*	2 860*G*	3 461*G*	6 110*G*	2 775
1894	2 825*G*	1 342*G*	777*G*	600*G*	1 059*G*	1 695*G*	3 390*G*	2 967*G*	3 390*G*	3 567*G*	4 980*G*	8 088*G*	2 890
1895	2 684*G*	1 286*G*	742*M*	918*G*	1 483*M*	2 048*M*	1 801*M*	2 543*M*	1 907*M*	3 638*M*	2 543*M*	3 920*M*	2 122
1896	2 437*M*	1 059*M*	600*M*	1 271*M*	2 543*M*	2 225*M*	2 225*G*	2 154*M*	2 154*M*	2 048*M*	2 472*M*	3 885*G*	2 089
1897	1 342*G*	918*G*	424*G*	563*G*	1 507*G*	1 801*G*	2 861*G*	3 108*G*	2 931*G*	2 931*G*	3 390*G*	3 532*G*	2 192
1898	3 284*G*	1 236*G*	849*G*	1 519*G*	1 872*G*	2 190*G*	3 214*G*	3 143*G*	1 907*G*	2 649*G*	3 955*G*	1 731*G*	2 296
1899	2 684*G*	777*G*	777*G*	883*F*	1 589*F*	2 154*F*	2 225*F*	3 284*F*	2 402*F*	2 649*F*	2 931*F*	2 260*F*	2 051
1900	1 589*F*	812*F*	530*F*	565*F*	342*F*	1 554*F*	2 190*F*	2 825*F*	2 154*F*	3 214*F*	2 967*F*	2 296*F*	1 837
1901	989*F*	671*F*	459*F*	459*F*	1 554*F*	1 731*F*	1 978*F*	2 437*F*	2 931*F*	2 755*F*	4 768*F*	3 037*F*	1 981
1902	4 238	1 271	812	1 695	2 119*	2 190	2 119	2 331	2 366	3 214	3 143	1 836	2 278
1903	1 130	848	494	565	1 377	1 942	3 108	3 178	2 790	2 755	4 009	5 474	2 306
1904	3 002	1 589	954	2 190	1 820	2 659	2 677	1 579	2 834	2 031	4 143	2 350	2 319
1905	1 418	777	578	442	1 932	1 594	1 248	2 207	1 722	2 688	2 267	1 265	1 512
1906	682	613	484	711	1 098	1 363	3 019	3 242	2 783	2 080	3 433	7 283	2 228
1907	3 995	1 376	1 013	652	2 132	2 989	4 225	2 955	3 904	4 583	3 866	2 896	2 882
1908	1 005	582	585	592	2 595	2 213	2 639	3 007	3 710	3 611	6 400	4 123	2 588
1909	4 040	3 315	1 155	1 110	2 420	4 870	3 300	4 090	3 560	3 665	11 300	17 300	5 010
Mean 20 Years	2 197	1 144	722	1 007	2 094	2 308	2 877	2 985	2 861	3 188	4 211	4 519	2 509
1910	5 050	2 870	1 625	3 140	5 220	3 310	4 420	4 400	4 339	4 398	4 490	5 964	4 102
Mean 21 Years	2 333	1 226	765	1 109	2 243	2 356	2 951	3 053	2 932	3 246	4 225	4 588	2 585
1911	1 640	2 310	881	1 689	2 790	2 881	2 609	.....	.....	.....	.....	.....	....
Mean 22 Years	2 301	1 275	770	1 134	2 267	2 379	2 935	.....	.....	.....	.....	.....	....

* Monthly means. *G* From Gamboa : Alhajuela ratio. *M* Mean of measured discharges. *F* From formulas.

TABLE 20.—Discharge, Alhajuela, in Cubic Feet per Second per Square Mile.
Water-shed Area = 427 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	5.02	2.56	2.15	2.07	7.61	9.51	9.59	10.09	12.16	11.50	7.03	10.49	7.48
1891	3.64	1.82	1.16	0.99	6.20	4.22	6.70	4.96	5.38	7.94	13.32	9.10	5.45
1892	2.73	2.15	1.90	3.97	9.84	6.20	13.23	10.84	7.78	10.59	12.91	10.83	7.75
1893	4.05	3.89	2.40	5.62	5.46	5.70	6.36	8.44	6.95	6.70	8.11	14.31	6.50
1894	6.62	3.14	1.82	1.41	2.48	3.97	7.94	6.95	7.94	8.35	11.67	18.94	6.77
1895	6.29	2.90	1.74	2.15	3.47	4.79	4.22	5.96	4.47	8.52	5.96	9.18	4.97
1896	5.70	2.48	1.41	2.98	5.96	5.21	5.21	5.05	5.05	4.79	5.79	9.10	4.90
1897	3.14	2.15	0.99	1.32	5.87	4.22	6.70	7.28	6.86	6.86	7.94	8.27	5.13
1898	7.69	2.90	1.99	3.56	4.39	5.13	7.53	7.36	4.47	6.20	9.26	4.05	5.38
1899	6.29	1.82	1.82	2.07	3.72	5.05	5.21	7.69	5.62	6.20	6.86	5.29	4.80
1900	3.72	1.90	1.24	1.32	3.14	3.64	5.13	6.61	5.05	7.52	6.95	5.38	4.30
1901	2.32	1.57	1.08	1.07	3.64	4.05	4.63	5.71	6.86	6.45	11.16	7.11	4.64
1902	9.93	2.98	1.90	3.97	4.96	5.13	4.96	5.46	5.51	7.53	7.36	4.30	5.34
1903	2.65	1.99	1.16	1.32	3.23	4.52	7.28	7.44	6.53	6.45	9.39	12.82	5.40
1904	7.03	3.72	2.23	5.13	4.26	6.22	6.27	3.70	6.64	4.76	9.70	5.50	5.43
1905	3.32	1.82	1.35	1.04	4.52	3.73	2.92	5.17	4.03	6.29	5.31	2.96	3.54
1906	1.60	1.41	1.02	1.67	2.57	3.19	7.07	7.59	6.52	4.87	8.04	17.06	5.22
1907	9.35	3.22	2.37	1.53	5.00	7.00	9.90	6.92	9.14	10.74	9.05	6.78	6.75
1908	2.35	1.36	1.37	1.39	6.08	5.18	6.18	7.04	8.69	8.46	14.99	9.66	6.06
1909	9.46	7.76	2.71	2.60	5.67	11.41	7.73	9.58	8.34	8.58	26.48	40.52	11.74
Mean 20 Years..	5.14	2.68	1.69	2.36	4.90	5.40	6.74	6.99	6.70	7.47	9.86	10.58	5.88
1910	11.83	6.72	3.81	7.36	12.22	7.75	10.36	10.31	10.16	10.30	10.51	13.97	9.61
Mean 21 Years..	5.46	2.87	1.79	2.60	5.25	5.52	6.91	7.15	6.87	7.60	9.90	10.74	6.06
1911	3.84	5.41	2.06	3.96	6.53	6.75	6.11	.....	.....	.....	.....	.....	.....
Mean 22 Years..	5.39	2.99	1.80	2.66	5.31	5.57	6.87	.....	.....	.....	.....	.....	.....

TABLE 21.—Discharge, Alhajuela, in Inches on Water-shed.

Water-shed Area = 427 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual total.
1890	5.788	2.666	2.479	2.309	8.773	10.610	11.056	11.629	13.566	13.260	7.843	12.098	102.077
1891	4.197	1.895	1.337	1.105	7.148	4.708	7.724	5.718	6.002	9.154	14.864	10.491	74.343
1892	3.147	2.319	2.190	4.429	11.344	6.917	15.253	12.498	8.680	12.209	14.401	12.486	105.873
1893	4.669	4.051	2.767	6.270	6.295	6.359	7.382	9.730	7.754	7.724	9.048	16.501	88.500
1894	7.632	3.270	2.098	1.573	2.859	4.429	9.154	8.013	8.859	9.627	13.017	21.834	92.365
1895	7.252	3.020	2.006	2.399	4.001	5.344	4.865	6.871	4.987	9.823	6.650	10.584	67.802
1896	6.583	2.675	1.626	3.325	6.871	5.813	6.007	5.822	5.634	5.522	6.460	10.491	66.829
1897	3.620	2.239	1.141	1.473	6.767	4.708	7.724	8.393	7.654	7.909	8.859	9.534	70.021
1898	8.806	3.020	2.294	3.972	5.061	5.724	8.681	8.485	4.987	7.148	10.331	4.669	73.238
1899	7.252	1.895	2.098	2.309	4.289	5.634	6.007	8.866	6.270	7.148	7.654	6.099	65.521
1900	4.289	1.979	1.430	1.473	3.620	4.061	5.914	7.621	5.634	9.781	7.754	6.203	58.659
1901	2.675	1.635	1.245	1.194	4.197	4.519	5.338	6.583	7.654	7.436	12.452	8.197	63.125
1902	11.448	3.103	2.190	4.429	5.718	5.724	5.718	6.295	6.181	8.681	8.212	4.957	72.656
1903	3.055	2.072	1.337	1.473	3.724	5.043	8.393	8.578	7.286	7.436	10.476	14.782	73.655
1904	8.105	4.012	2.571	5.724	4.911	6.940	7.229	4.266	7.408	5.488	10.822	6.341	73.817
1905	3.828	1.895	1.556	1.160	5.211	4.162	3.366	5.960	4.496	7.252	5.924	3.413	48.223
1906	1.845	1.500	1.176	1.863	2.963	3.559	8.151	8.750	7.274	5.615	8.970	19.666	71.332
1907	10.780	3.353	2.732	1.707	5.764	7.810	11.414	7.978	10.197	12.384	10.097	7.817	92.033
1908	2.709	1.467	1.579	1.551	7.010	5.779	7.125	8.116	9.695	9.753	16.728	11.137	82.649
1909	10.906	8.081	3.124	2.901	6.537	12.731	8.912	11.045	9.305	9.892	29.546	46.714	159.694
Mean 20 Years..	5.932	2.807	1.949	2.632	5.653	6.029	7.768	8.061	7.476	8.607	11.005	12.201	80.120
1910	13.636	6.998	4.393	8.212	14.092	8.647	11.942	11.882	11.336	11.870	11.722	16.107	130.837
Mean 21 Years..	6.299	3.007	2.065	2.898	6.055	6.153	7.967	8.243	7.660	8.762	11.040	12.386	82.535
1911	4.427	5.634	2.375	4.418	7.528	7.531	7.044	.....	.....	.....	.....	.....	.....
Mean 22 Years..	6.214	3.126	2.079	2.967	6.122	6.216	7.924	.....	.....	.....	.....	.....	.....

TABLE 22.—Discharge, Gamboa, in Cubic Feet per Second.

Water-shed Area = 559 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	2 755	1 342*m*	1 095*m*	1 059	4 273	5 333	5 403*m*	5 686*m*	6 816*m*	6 498*m*	3 956*m*	5 898*m*	4 176
1891	2 048*m*	954*m*	600*m*	530*m*	3 461*m*	2 366*m*	3 779*m*	2 790*m*	3 037*m*	4 450*m*	7 487*m*	5 121*m*	3 052
1892	1 519*m*	1 095*m*	989*m*	2 048*m*	5 509	3 496	7 416	6 075	4 379	5 923	7 240	6 075	4 314
1893	2 296	2 013	1 236	2 931	3 037	3 214	3 567	4 732	3 885	3 744	4 556	8 052	3 605
1894	3 708	625	954	742	1 377	2 225	4 450	3 885	4 450	4 697*m*	6 534*m*	10 630*m*	3 690
1895	3 532	1 483	1 095*m*	1 130	3 461	3 178	3 073	4 061	3 390	4 944	3 532	3 779	3 055
1896	2 225	1 130	636*m*	1 342*m*	4 450*m*	4 132	2 931	2 331*m*	2 719*m*	2 684*m*	3 249	5 121*m*	2 746
1897	1 760*m*	1 130	494*m*	671	3 284*m*	2 366*m*	3 779*m*	4 097*m*	3 850*m*	3 850	4 450	4 626	2 863
1898	4 309	1 483	1 024	1 836	2 472	2 861	4 238	4 132	2 543	3 461	5 192	2 296	2 987
1899	3 496	954	954	1 059	1 907	2 472	2 825	4 238	3 178	3 390	3 744	2 649	2 572
1900	1 695	812	530	565	1 342	1 942	3 355	3 567	3 073	4 520	4 061	2 684	2 346
1901	1 024	671	459	459	1 766	2 048	2 190	2 861	3 638	3 702	6 746	3 143	2 392
1902	1 227	1 589	1 024	1 942	2 896	2 896	3 002	2 790	2 931	4 415	4 732	2 190	2 970
1903	1 625	918	706	706	1 519	2 154	3 320	4 344	4 697	3 956	5 721	7 028	3 058
1904	4 132	2 402	1 695	2 861	2 275	3 073	3 271	2 142	3 822	2 974	4 561	2 711	2 993
1905	1 687	1 032	844	703	2 183	2 091	1 684	2 613	2 231	3 480	2 585	1 835	1 914
1906	1 020	783	610	849	1 916	1 949	3 835	4 137	3 781	2 782	5 252	7 756	2 889
1907	2 935	1 297	1 011	687	1 446	2 879	5 560	3 890	5 140	6 030	5 090	3 810	3 315
1908	1 230	710	710	720	3 410	2 910	3 470	3 960	4 880	4 750	8 420	5 820	3 416
1909	4 930	4 040	1 410	1 350	3 180	6 410	4 340	5 380	4 680	4 820	14 870	22 760	6 514
1910	6 160	3 500	1 980	3 830	6 870	4 360	5 820	5 790	5 710	5 790	5 652	8 078	5 295
Mean 21 Years..	2 824	1 427	955	1 334	2 954	3 064	3 872	3 976	3 944	4 327	5 601	5 812	3 341
1911	1 766	2 362	976	1 756	3 178	3 020	3 516	.....	.....	.....	.....	.....	.....
Mean 22 Years..	2 776	1 469	956	1 353	2 962	3 062	3 856	.....	.....	.....	.....	.....	.....

m Denotes mean of all gaugings during month.

TABLE 23.—Discharge, Gamboa, in Cubic Feet per Second per Square Mile.

Water-shed Area = 559 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	4.92	2.40	1.96	1.90	7.64	9.54	9.67	10.18	12.20	11.61	7.07	10.54	7.47
1891	3.66	1.71	1.07	0.95	6.19	4.23	6.76	4.99	5.43	7.96	13.40	9.16	5.46
1892	2.72	1.96	1.77	3.66	9.86	6.25	13.27	10.88	7.83	10.60	12.95	10.88	7.72
1893	4.11	3.60	2.21	5.24	5.43	5.75	6.38	8.47	6.95	6.70	8.15	14.40	6.45
1894	6.63	1.12	1.71	1.33	2.46	3.98	7.96	6.95	7.96	8.40	11.69	19.02	6.60
1895	6.32	2.65	1.96	2.02	6.20	5.68	5.50	7.27	6.06	8.84	6.32	6.76	5.47
1896	3.98	2.02	1.14	2.40	7.96	7.40	5.25	4.17	4.86	4.80	5.81	9.16	4.91
1897	3.15	2.02	0.88	1.20	5.87	4.23	6.76	7.32	6.88	6.88	7.96	8.27	5.12
1898	7.70	2.65	1.83	3.28	4.42	5.12	7.58	7.39	4.55	6.20	9.29	4.10	5.34
1899	6.25	1.71	1.71	1.89	3.41	4.42	5.05	7.58	5.68	6.06	6.70	4.74	4.60
1900	3.03	1.46	0.95	1.01	2.40	3.47	6.00	6.38	5.50	8.08	7.26	4.80	4.20
1901	1.83	1.20	0.82	0.82	3.16	3.66	3.92	5.12	6.50	6.62	12.07	5.62	4.28
1902	9.35	2.84	1.83	3.47	5.18	5.18	5.37	4.99	5.24	7.90	8.47	3.92	5.31
1903	2.91	1.65	1.27	1.27	2.72	3.85	5.94	7.77	8.40	7.08	10.23	12.57	5.47
1904	7.39	4.30	3.03	5.12	4.07	5.50	5.85	3.83	6.84	5.32	8.16	4.85	5.36
1905	3.02	1.84	1.51	1.26	3.91	3.74	3.01	4.67	3.99	6.23	4.62	3.28	3.42
1906	1.82	1.40	1.09	1.52	3.43	3.49	6.86	7.40	6.76	4.98	9.39	13.88	5.17
1907	5.25	2.32	1.81	1.23	2.59	5.15	9.95	6.96	9.20	10.78	9.10	6.82	5.93
1908	2.20	1.27	1.27	1.29	6.10	5.20	6.20	7.08	8.73	8.50	15.05	10.41	6.11
1909	8.82	7.22	2.52	2.42	5.69	11.46	7.76	9.62	8.37	8.62	26.60	40.71	11.65
1910	11.01	6.26	3.54	6.85	12.29	7.80	10.41	10.36	10.20	10.36	10.11	14.42	9.47
Mean 21 Years.	5.05	2.55	1.71	2.39	5.28	5.48	6.93	7.11	7.05	7.74	10.02	10.40	5.98
1911	3.16	4.22	1.75	3.14	5.68	5.40	6.29	.....	.....	.....	.....	.....	.....
Mean 22 Years.	4.96	2.63	1.71	2.42	5.30	5.48	6.90	.....	.....	.....	.....	.....	.....

TABLE 24.—Discharge, Gamboa, in Inches, on Water-shed.

Water-shed Area = 559 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual total.
1890	5.672	2.499	2.260	2.120	8.808	10.644	11.148	11.736	13.610	13.382	7.888	12.154	101.921
1891	4.220	1.781	1.234	1.060	7.136	4.719	7.794	5.753	6.058	9.177	14.950	10.560	74.442
1892	3.136	2.114	2.041	4.083	11.367	6.973	15.297	12.546	8.736	12.220	14.445	12.546	105.504
1893	4.738	3.749	2.548	5.846	6.260	6.415	7.355	9.765	7.754	7.724	9.093	16.600	87.847
1894	7.644	1.166	1.971	1.484	2.836	4.440	9.177	8.013	8.881	9.684	13.039	21.924	90.259
1895	7.286	2.759	2.260	2.254	7.148	6.337	6.341	8.382	6.761	10.192	7.051	7.794	74.565
1896	4.589	2.179	1.314	2.678	9.177	8.256	6.053	4.808	5.422	5.534	6.482	10.560	67.052
1897	3.632	2.103	1.015	1.339	6.767	4.719	7.794	8.439	7.676	7.932	8.881	9.534	69.831
1898	8.877	2.759	2.110	3.659	5.096	5.712	8.739	8.520	5.076	7.148	10.365	4.727	72.788
1899	7.206	1.781	1.971	2.109	3.931	4.931	5.822	8.739	6.337	6.987	7.475	5.465	62.754
1900	3.493	1.520	1.095	1.127	2.767	3.871	6.917	7.355	6.136	9.315	8.100	5.534	57.230
1901	2.110	1.250	0.945	0.915	3.643	4.083	4.519	5.903	7.252	7.632	13.467	6.479	58.198
1902	10.780	2.957	2.110	3.871	5.972	5.779	6.191	5.753	5.846	9.108	9.450	4.519	72.336
1903	3.355	1.718	1.464	1.417	3.136	4.295	6.848	8.958	9.372	8.162	11.413	14.494	74.632
1904	8.520	4.638	3.493	5.712	4.692	6.136	6.744	4.416	7.631	6.133	9.104	5.592	72.811
1905	3.482	1.916	1.741	1.406	4.508	4.173	3.470	5.384	4.452	7.183	5.155	3.781	46.651
1906	2.098	1.458	1.257	1.696	3.954	3.894	7.909	8.531	7.542	5.741	10.476	16.006	70.562
1907	6.053	2.416	2.087	1.372	2.986	5.746	11.470	8.024	10.264	12.428	10.153	7.863	80.862
1908	2.536	1.370	1.464	1.439	7.033	5.802	7.148	8.162	9.740	9.800	16.795	12.002	83.291
1909	10.168	7.518	2.905	2.700	6.560	12.786	8.946	11.091	9.338	9.938	29.680	46.932	158.562
1910	12.692	6.519	4.081	7.643	14.169	8.702	12.002	11.942	11.380	11.942	11.281	16.624	128.977
Mean 21 Years..	5.823	2.675	1.970	2.663	6.093	6.115	7.985	8.201	7.870	8.922	11.178	11.985	81.480
1911	3.643	4.394	2.018	3.503	6.548	6.025	7.252	.....	.....	.....	.....	.....	.....
Mean 22 Years..	5.724	2.753	1.972	2.702	6.068	6.111	7.952	.....	.....	.....	.....	.....	.....

TABLE 25.—Discharge, Bohio, in Cubic Feet per Second.

Water-shed Area = 779 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
1890	4 132*	1 554*G*	1 271*G*	1 236*G*	6 569*G*	8 193*G*	8 300*G*	8 759*G*	10 489*G*	9 995*G*	6 075*G*	9 076*G*	6 304
1891	2 684*M*	1 377*M*	600*M*	600*M*	5 333*G*	3 214*M*	4 803*M*	4 521*M*	4 661*G*	6 851*G*	11 514*G*	7 876*G*	4 503
1892	2 331*G*	1 271*G*	1 165*G*	2 366*G*	8 476*G*	5 439*G*	11 407*G*	9 359*G*	6 746*G*	9 112*G*	11 125*G*	9 359*G*	6 513
1893	3 532*G*	2 331*G*	1 448*G*	3 426*G*	4 662*G*	4 944*M*	7 628*M*	10 772*M*	7 664*M*	8 582*M*	11 443*M*	18 541*M*	7 081
1894	7 003*M*	1 801*M*	1 045*M*	777*M*	3 355*M*	4 278*M*	6 216*M*	6 498*M*	7 452*M*	9 359*M*	12 714*M*	12 573*M*	6 098
1895	4 556*M*	1 377*M*	1 059*M*	1 059*M*	4 167*M*	4 309*M*	4 591*M*	6 922*M*	5 792*M*	6 922*M*	6 075*M*	6 957*M*	4 483
1896	4 732*M*	1 554*M*	883*M*	1 695*M*	4 944*M*	4 203*M*	3 143*M*	3 496*M*	5 933*M*	6 357*M*	7 593*M*	6 039*M*	4 214
1897	2 048*M*	1 589*M*	706*M*	883*M*	7 982*M*	4 203*M*	5 121*M*	7 275*M*	6 887*M*	7 487*M*	6 957*M*	6 816*M*	4 830
1898	5 651*F*	1 695*F*	1 095*F*	2 013*F*	2 684*F*	3 355*F*	6 569*F*	5 298*F*	3 214*F*	5 050*F*	7 840*F*	2 861*F*	3 944
1899	3 850*F*	1 731*F*	989*F*	1 095*F*	2 048*F*	3 143*F*	3 956*F*	5 721*F*	4 238*F*	4 944*F*	5 296*F*	3 602*F*	3 384
1900	2 119*F*	1 059*F*	742*F*	706*F*	1 625*F*	2 861*F*	5 615*F*	5 262*F*	4 768*F*	6 993*F*	6 605*F*	3 744*F*	3 508
1901	1 483*F*	989*F*	706*F*	365*F*	2 296*F*	2 790*F*	3 178*F*	4 309*F*	5 792*F*	6 322*F*	13 244*F*	4 591*F*	3 855
1902	11 231*F*	2 084*F*	1 271*F*	2 154*F*	4 273*F*	3 178*F*	3 037*F*	3 390*F*	3 920*F*	5 933*F*	6 675*F*	3 002*F*	4 179
1903	1 625*F*	1 024*F*	636*F*	565*F*	1 519*F*	2 543*F*	4 026*F*	5 474*F*	5 686*F*	5 580*F*	8 335*F*	10 489*F*	3 958
1904	4 944*F*	2 331*F*	1 413*F*	3 638*F*	3 037	4 732	4 520	3 178	6 674	4 909	7 628	4 132	4 261
1905	1 942	989	706	600	2 825	2 649	1 907	3 884	3 955	6 957	4 732	2 860	2 834
1906	1 338	864	611	1 002	2 152	2 672	5 920	6 356	4 557	4 078	7 826	10 654	4 078
1907	3 339	1 396	1 002	674	1 854	3 658	4 662	3 952	5 664	7 842	5 443	3 358	3 570
1908	1 281	681	586	666	3 894	3 131	3 626	5 491	5 599	5 558	9 515	5 046	3 821
1909	4 390	3 580	1 325	1 185	3 225	7 010	5 630	6 600	5 550	8 230	21 380	19 800	7 325
1910	6 160	2 894	3 080	1 875	3 760	7 470	5 950	9 350	8 010	7 840	.....	.....	.....
Mean 20 Years.	3 714	1 564	963	1 335	3 846	4 025	5 193	5 826	5 807	6 853	8 901	5 769	4 637
Mean 21 Years	3 830	1 627	1 064	1 361	3 842	4 189	5 229	5 994	5 912	6 900	.....	.....	.....

* Monthly Mean. *G* Gamboa: Bohio ratio. *M* Mean of Measured Discharges. *F* From Formulas.

TABLE 26.—Discharge, Bohio, in Cubic Feet per Second per Square Mile.

Water-shed Area = 779 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	5.31	2.00	1.63	1.59	8.43	10.51	10.65	11.23	13.47	12.82	7.80	11.64	8.09
1891	3.45	1.77	0.77	0.77	6.84	4.13	6.16	5.80	5.98	8.79	14.78	10.11	5.78
1892	2.99	1.63	1.50	3.04	10.88	6.98	14.65	12.01	8.66	11.69	14.28	12.01	8.36
1893	4.54	2.99	1.86	4.40	5.99	6.35	9.79	13.83	9.84	11.02	14.69	23.81	9.09
1894	9.07	2.31	1.34	1.00	4.31	5.48	7.98	8.34	9.56	12.01	16.32	16.14	7.82
1895	5.85	1.77	1.36	1.36	5.35	5.53	5.90	8.88	7.44	8.88	7.80	8.93	5.75
1896	6.08	2.00	1.13	2.18	6.35	5.40	4.04	4.49	7.62	8.16	9.75	7.75	5.41
1897	2.63	2.04	0.91	1.13	10.24	5.40	6.58	9.34	8.84	9.61	8.93	8.75	6.20
1898	7.26	2.18	1.41	2.59	3.45	4.31	8.44	6.80	4.13	6.48	10.07	3.67	5.07
1899	4.94	2.22	1.27	1.41	2.63	4.04	5.08	7.34	5.44	6.35	6.80	4.62	4.34
1900	2.72	1.36	0.95	0.91	2.09	3.67	7.20	6.75	6.12	8.98	8.48	4.80	4.50
1901	1.90	1.27	0.91	0.47	2.95	3.58	4.08	5.53	7.43	8.12	17.00	5.89	4.93
1902	14.42	2.68	1.63	2.77	5.49	4.08	3.90	4.35	5.03	7.61	8.57	3.85	5.36
1903	2.09	1.31	0.82	0.73	1.95	3.26	5.17	7.03	7.30	7.16	10.69	13.47	5.08
1904	6.34	2.99	1.81	4.67	3.90	6.08	5.80	4.08	8.57	6.30	9.79	5.30	5.47
1905	2.49	1.27	0.91	0.77	3.63	3.40	2.45	4.99	5.08	8.92	6.08	3.67	3.64
1906	1.72	1.11	0.78	1.29	2.76	3.43	7.60	8.16	7.00	5.24	10.04	13.68	5.23
1907	4.29	1.79	1.29	0.86	2.38	4.70	5.99	5.08	7.27	10.07	6.99	4.31	4.58
1908	1.64	0.87	0.75	0.86	5.00	4.02	4.65	7.05	7.19	7.13	12.21	6.48	4.82
1909	5.64	4.60	1.70	1.52	4.14	9.00	7.23	8.47	7.12	10.56	27.44	25.42	9.40
1910	7.91	3.72	3.95	2.41	4.83	9.59	7.64	12.00	10.28	10.07	.....	.....	.....
Mean 20 Years..	4.77	2.01	1.24	1.72	4.94	5.17	6.67	7.48	7.45	8.80	11.43	9.72	5.95
Mean 21 Years..	4.92	2.09	1.37	1.75	4.93	5.38	6.71	7.69	7.59	8.86	.....	.....	.....

TABLE 27.—Discharge, Bohio, in Inches, on Water-shed.

Water-shed Area = 779 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual total.
1890	6.122	2.083	1.879	1.774	9.719	11.722	12.280	12.946	15.027	14.782	8.702	13.418	110.454
1891	3.977	1.843	0.888	0.859	7.886	4.608	7.102	6.687	6.672	10.134	16.488	11.652	78.796
1892	3.447	1.758	1.729	3.392	12.546	7.788	16.890	13.842	9.662	13.478	15.928	13.842	114.302
1893	5.234	3.111	2.144	4.909	6.906	7.085	11.287	15.946	10.978	12.704	16.389	27.451	124.147
1894	10.457	2.405	1.545	1.116	4.967	6.114	9.200	9.615	10.666	13.842	18.212	18.608	106.749
1895	6.744	1.843	1.568	1.517	6.168	6.170	6.802	10.248	8.301	10.238	8.702	10.295	78.586
1896	7.010	2.157	1.303	2.432	7.321	6.025	4.658	5.176	8.502	9.408	10.878	8.935	73.805
1897	3.032	2.124	1.049	1.261	11.804	6.025	7.586	10.768	9.863	11.079	9.963	10.088	84.642
1898	8.370	2.270	1.626	2.890	3.977	4.809	9.730	7.840	4.608	7.471	11.237	4.231	69.059
1899	5.695	2.312	1.464	1.573	3.032	4.507	5.857	8.462	6.069	7.321	7.587	5.326	59.205
1900	3.136	1.416	1.095	1.015	2.410	4.095	8.301	7.782	6.828	10.353	9.461	5.534	61.426
1901	2.190	1.322	1.049	0.524	3.401	3.994	4.704	6.375	8.290	9.361	18.970	6.791	66.971
1902	16.624	2.791	1.879	3.090	6.329	4.552	4.496	5.015	5.612	8.773	9.562	4.439	73.162
1903	2.410	1.364	0.945	0.814	2.248	3.637	5.960	8.105	8.145	8.255	11.929	15.527	69.339
1904	7.309	3.225	2.087	5.210	4.496	6.783	6.687	4.704	9.562	7.263	10.923	6.110	74.359
1905	2.871	1.322	1.049	0.859	4.185	3.793	2.825	5.753	5.668	10.284	6.783	4.231	49.623
1906	1.983	1.156	0.899	1.439	3.182	3.827	8.762	9.408	7.810	6.041	11.204	15.768	71.479
1907	4.946	1.864	1.487	0.960	2.744	5.244	6.906	5.857	8.111	11.607	7.799	4.969	62.494
1908	1.891	0.938	0.865	0.960	5.764	4.485	5.361	8.128	8.022	8.220	13.621	7.471	65.726
1909	6.502	4.790	1.960	1.696	4.773	10.041	8.335	9.765	7.944	12.176	30.614	29.304	127.900
1910	9.119	3.874	4.554	2.689	5.568	10.700	8.808	13.830	11.468	11.607	.....	.....	.....
Mean 20 Years	5.498	2.105	1.426	1.914	5.693	5.765	7.686	8.621	8.317	10.140	12.748	11.200	81.113
Mean 21 Years	5.670	2.189	1.574	1.951	5.687	6.000	7.741	8.869	8.467	10.209	.....	.....	.....

TABLE 28.—Discharge, Gatun, in Cubic Feet per Second.
Water-shed Area = 1 320 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	6 694	2 518	2 059	2 002	10 624	13 273	13 446	14 190	16 992	16 102	9 842	14 703	10 204
1891	4 348	2 231	972	972	8 640	5 207	7 781	7 324	7 551	11 099	18 651	12 759	7 295
1892	3 776	2 059	1 887	3 833	13 731	8 811	18 479	15 162	10 928	14 741	18 022	15 162	10 549
1893	5 722	3 776	2 346	5 550	7 552	8 009	12 357	17 450	12 416	13 903	18 538	30 037	11 471
1894	11 442	2 918	1 693	1 259	5 435	6 922	10 070	10 527	12 072	15 162	20 597	20 368	9 872
1895	7 381	2 231	1 716	1 716	6 750	6 981	7 437	11 214	9 383	11 214	9 842	11 270	7 261
1896	7 666	2 518	1 430	2 745	8 009	6 809	5 092	5 664	9 612	10 298	12 301	9 783	6 827
1897	3 318	2 574	1 144	1 430	12 931	6 809	8 296	11 786	11 157	12 129	11 271	11 042	7 824
1898	9 155	2 746	1 774	3 261	4 348	5 435	10 642	8 583	5 207	8 181	12 701	4 635	6 389
1899	6 237	2 804	1 602	1 774	3 318	5 092	6 409	9 268	6 866	8 009	8 580	5 835	5 483
1900	3 433	1 716	1 202	1 144	2 632	4 635	9 096	8 524	7 724	11 329	10 700	6 065	5 683
1901	2 402	1 602	1 144	915	3 720	4 520	5 148	6 981	9 383	10 242	21 455	7 437	6 246
1902	18 194	3 376	2 059	3 490	4 318	5 148	4 920	5 492	6 350	9 612	10 814	4 863	6 553
1903	2 632	1 659	1 030	915	2 461	4 120	6 522	8 868	9 211	9 034	13 503	16 992	6 412
1904	8 009	3 776	2 289	5 894	4 910	7 652	7 308	5 139	10 792	7 938	12 334	6 681	6 894
1905	3 140	1 599	1 142	970	4 568	4 283	3 083	6 283	6 395	11 250	7 652	4 624	4 582
1906	2 164	1 397	988	1 620	3 479	4 321	8 472	10 277	8 824	6 594	12 654	17 237	6 501
1907	5 399	2 257	1 620	1 090	2 997	5 915	7 135	5 941	8 999	13 479	9 299	5 602	5 811
1908	1 976	1 046	851	892	5 046	5 025	5 744	8 886	8 708	8 859	14 408	8 017	5 788
1909	6 505	5 260	2 150	2 215	4 215	9 880	9 920	10 980	9 910	12 590	28 470	25 800	10 658
1910	11 740	5 080	3 160	6 090	10 270	11 060	13 800	13 020	11 637	12 710	17 312	21 820	11 472
Mean 21 Years..	6 254	2 626	1 631	2 370	6 188	6 662	8 627	9 598	9 529	11 165	14 236	12 416	7 608
1911	3 636	2 999	1 439	2 071	5 952	6 488	6 905	.....	.....	.....	.....	.....	.....
Mean 22 Years..	6 135	2 643	1 622	2 357	6 177	6 654	8 548	.....	.....	.....	.....	.....	.....

Jan., 1890, to Apr., 1907, from (Bohio Discharge) × (1.62).
May, 1907, to Apr., 1908, from (Bohio + Trinidad + Gatun River) × (1.10).

TABLE 29.—Discharge, Gatun, in Cubic Feet per Second per Square Mile.

Water-shed Area = 1 320 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	5.07	1.91	1.56	1.52	8.05	10.06	10.19	10.74	12.88	12.20	7.46	11.14	7.73
1891	3.29	1.69	0.74	0.74	6.54	3.95	5.90	5.55	5.72	8.40	14.13	9.67	5.52
1892	2.86	1.56	1.43	2.90	10.40	6.68	14.00	11.49	8.28	11.17	13.66	11.48	7.99
1893	4.34	2.86	1.78	4.20	5.72	6.07	9.36	13.22	9.41	10.53	14.05	22.72	8.68
1894	8.67	2.21	1.28	0.95	4.12	5.25	7.63	7.98	9.14	11.49	15.61	15.43	7.48
1895	5.59	1.69	1.30	1.30	5.11	5.29	5.64	8.49	7.11	8.49	7.46	8.54	5.50
1896	5.81	1.91	1.08	2.08	6.07	5.16	3.86	4.29	7.28	7.80	9.32	7.42	5.17
1897	2.51	1.95	0.87	1.08	9.80	5.16	6.28	8.93	8.45	9.19	8.54	8.36	5.93
1898	6.93	2.08	1.34	2.47	3.29	4.12	8.06	6.50	3.95	6.20	9.62	3.51	4.84
1899	4.72	2.12	1.21	1.34	2.51	3.86	4.86	7.02	5.20	6.07	6.50	4.42	4.15
1900	2.60	1.30	0.91	0.87	2.00	3.51	6.89	6.46	5.86	8.53	8.10	4.60	4.31
1901	1.82	1.21	0.87	0.69	2.82	3.43	3.90	5.29	7.11	7.76	16.26	5.64	4.73
1902	13.78	2.56	1.56	2.64	3.27	3.90	3.73	4.16	4.81	7.28	8.19	3.69	4.96
1903	1.99	1.26	0.78	0.69	1.86	3.12	4.94	6.72	6.98	6.84	10.23	12.88	4.86
1904	6.07	2.86	1.73	4.46	3.72	5.80	5.54	3.89	8.18	6.02	9.34	5.06	5.22
1905	2.38	1.21	0.87	0.73	3.46	3.25	2.34	4.76	4.85	8.52	5.80	3.50	3.47
1906	1.64	1.06	0.75	1.23	2.64	3.27	6.42	7.78	6.69	5.00	9.58	13.07	4.93
1907	4.09	1.71	1.23	0.83	2.27	4.48	5.40	4.50	6.82	10.21	7.04	4.24	4.40
1908	1.50	0.79	0.64	0.68	3.82	3.81	4.35	6.73	6.60	6.72	10.91	6.08	4.38
1909	4.93	3.98	1.63	1.68	3.19	7.48	7.52	8.32	7.51	9.54	21.58	19.55	8.07
1910	8.90	3.85	2.39	4.61	7.78	8.38	10.45	9.86	8.82	9.63	13.11	16.53	8.69
Mean 21 Years..	4.74	1.98	1.24	1.80	1.69	5.05	6.54	7.27	7.22	8.46	10.78	9.41	5.76
1911	2.75	2.27	1.09	1.57	4.51	4.92	5.23	....	....	....	....	....	....
Mean 22 Years..	4.65	2.00	1.23	1.78	4.68	5.04	6.48	....	....	....	....	....	....

TABLE 30.—Discharge, Gatun, in Inches, on Water-shed.

Water-shed Area = 1 320 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual total.
1890	5.845	1.989	1.799	1.696	9.281	11.226	11.748	12.384	14.368	14.070	8.323	12.844	105.573
1891	3.793	1.760	0.853	0.826	7.540	4.407	6.802	6.399	6.382	9.684	15.763	11.148	75.357
1892	3.297	1.682	1.649	3.236	11.990	7.453	16.140	13.248	9.238	12.877	15.242	13.236	109.288
1893	5.004	2.978	2.052	4.686	6.595	6.772	10.791	15.242	10.499	12.143	15.675	26.194	118.631
1894	9.996	2.301	1.476	1.060	4.750	5.857	8.797	9.200	10.197	13.248	17.412	17.786	102.080
1895	6.445	1.760	1.499	1.450	5.891	5.902	6.502	9.788	7.933	9.788	8.323	9.846	75.127
1896	6.698	2.060	1.245	2.321	6.998	5.757	4.450	4.946	8.122	8.993	10.398	8.554	70.542
1897	2.894	2.031	1.003	1.205	11.298	5.757	7.240	10.295	9.428	10.595	9.528	9.638	80.912
1898	7.990	2.166	1.545	2.756	3.793	4.597	9.292	7.494	4.407	7.148	10.733	4.047	65.968
1899	5.442	2.197	1.395	1.495	2.894	4.307	5.603	8.093	5.802	6.998	7.252	5.096	56.574
1900	2.998	1.354	1.049	0.971	2.306	3.916	7.943	7.448	6.538	9.892	9.037	5.303	58.755
1901	2.098	1.260	1.003	0.770	3.251	3.827	4.496	6.099	7.933	8.946	18.142	6.502	64.327
1902	15.886	2.666	1.799	2.945	3.770	4.351	4.300	4.796	5.367	8.393	9.138	4.254	67.665
1903	2.294	1.312	0.899	0.770	2.144	3.481	5.695	7.747	7.788	7.886	11.413	14.846	66.277
1904	6.998	3.085	1.994	4.976	4.289	6.471	6.387	4.485	9.126	6.940	10.421	5.834	71.006
1905	2.744	1.260	1.003	0.814	3.989	3.626	2.698	5.488	5.411	9.823	6.471	4.035	47.362
1906	1.891	1.104	0.865	1.372	3.044	3.648	7.402	8.969	7.464	5.764	10.688	15.067	67.280
1907	4.715	1.781	1.418	0.926	2.617	4.998	6.226	5.188	7.609	11.771	7.855	4.888	59.992
1908	1.729	0.852	0.738	0.759	4.404	4.251	5.015	7.759	7.364	7.747	12.171	7.010	59.799
1909	5.684	4.144	1.879	1.874	3.678	8.345	8.670	9.592	8.379	10.999	24.078	22.540	109.862
1910	10.261	4.009	2.755	5.143	8.969	9.350	12.050	11.367	9.840	11.102	14.631	19.056	118.533
Mean 21 Years..	5.462	2.083	1.425	2.002	5.404	5.633	7.535	8.382	8.057	9.753	12.033	10.844	78.613
1911	3.170	2.364	1.257	1.752	5.200	5.489	6.030	.....	.....	.....	.....	.....	.....
Mean 22 Years..	5.358	2.096	1.417	1.991	5.396	5.627	7.467	.....	.....	.....	.....	.....	.....

TABLE 31.—Discharge, Trinidad River.

(Above Lagartera Station, Water-shed Area = 314 Sq. Miles.)

Cubic Feet per Second.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
1907	....	....	....	....	556	1 097	985	845	1 334	3 144	2 076	1 135	....
1908	356	186	131	105	946	1 118	1 027	1 939	1 853	1 825	2 359	1 631	1 123
1909	1 398	988	365	210	782	1 870	1 760	2 155	1 940	2 580	2 660	2 470	1 598
1910	1 940	840	565	1 250	1 880	1 980	2 000	1 500	1 900	2 000	2 500	2 200	1 713
1911	1 000	300	170	190	....	....	....	....	....	....	....	....	....
Means (4 Years)	1 174	578	308	439	1 041	1 516	1 443	1 610	1 757	2 387	2 399	1 859	1 376

Cubic Feet per Second per Square Mile.

1907	....	....	....	....	1.77	3.49	3.14	2.69	4.25	10.02	6.49	3.61	....
1908	1.13	0.59	0.42	0.33	3.01	3.56	3.27	6.18	5.90	5.81	7.51	5.19	3.58
1909	4.45	3.15	1.16	0.67	2.49	5.96	5.60	6.86	6.18	8.22	8.47	7.87	5.09
1910	6.18	2.67	1.80	3.98	5.99	6.31	6.37	4.78	6.05	6.37	7.96	7.01	5.46
1911	3.18	0.96	0.54	0.60	....	....	....	....	....	....	....	....	....
Means (4 Years)	3.74	1.84	0.98	1.40	3.32	4.83	4.60	5.13	5.60	7.60	7.61	5.92	4.38

Inches on Water-shed.

													Total.
1907	.....	.....	.....	.....	2.041	3.894	3.620	3.101	4.742	11.552	7.241	4.162	....
1908	1.303	0.636	0.484	0.368	3.470	3.972	3.770	7.125	6.583	6.698	8.379	5.982	48.771
1909	5.130	3.280	1.337	0.748	2.871	6.650	6.456	7.909	6.895	9.477	9.450	9.073	69.276
1910	7.125	2.780	2.075	4.440	6.906	7.040	7.344	5.511	6.750	7.344	8.881	8.082	74.278
1911	3.666	1.000	0.623	0.669	....	....	....	....	....	....	....	....	....
Means (4 Years)	4.306	1.924	1.130	1.556	3.822	5.389	5.298	5.912	6.242	8.768	8.488	6.825	59.660

Station removed to ¼ mile below junction of Siri and Trinidad branch on October 7th, 1910.

TABLE 32.—Discharge, Gatun River.

(Above Monte Lirio—old station. Water-shed Area = 127 Sq. Miles.)

Cubic Feet per Second.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
1907	....	....	....	....	315	622	839	604	1 183	1 268	935	600	....
1908	159	84	57	40	533	460	708	788	675	983	1 821	627	578
1909	615	470	157	114	454	1 170	848	780	930	1 160	2 450	2 600	979
1910	805	440	270	550	950	1 140	1 400	1 300	1 160	1 290	1 600	2 500	1 117
Means (3 and 4 Years).	526	331	161	235	563	848	949	868	987	1 175	1 702	1 582	827

Cubic Feet per Second per Square Mile.

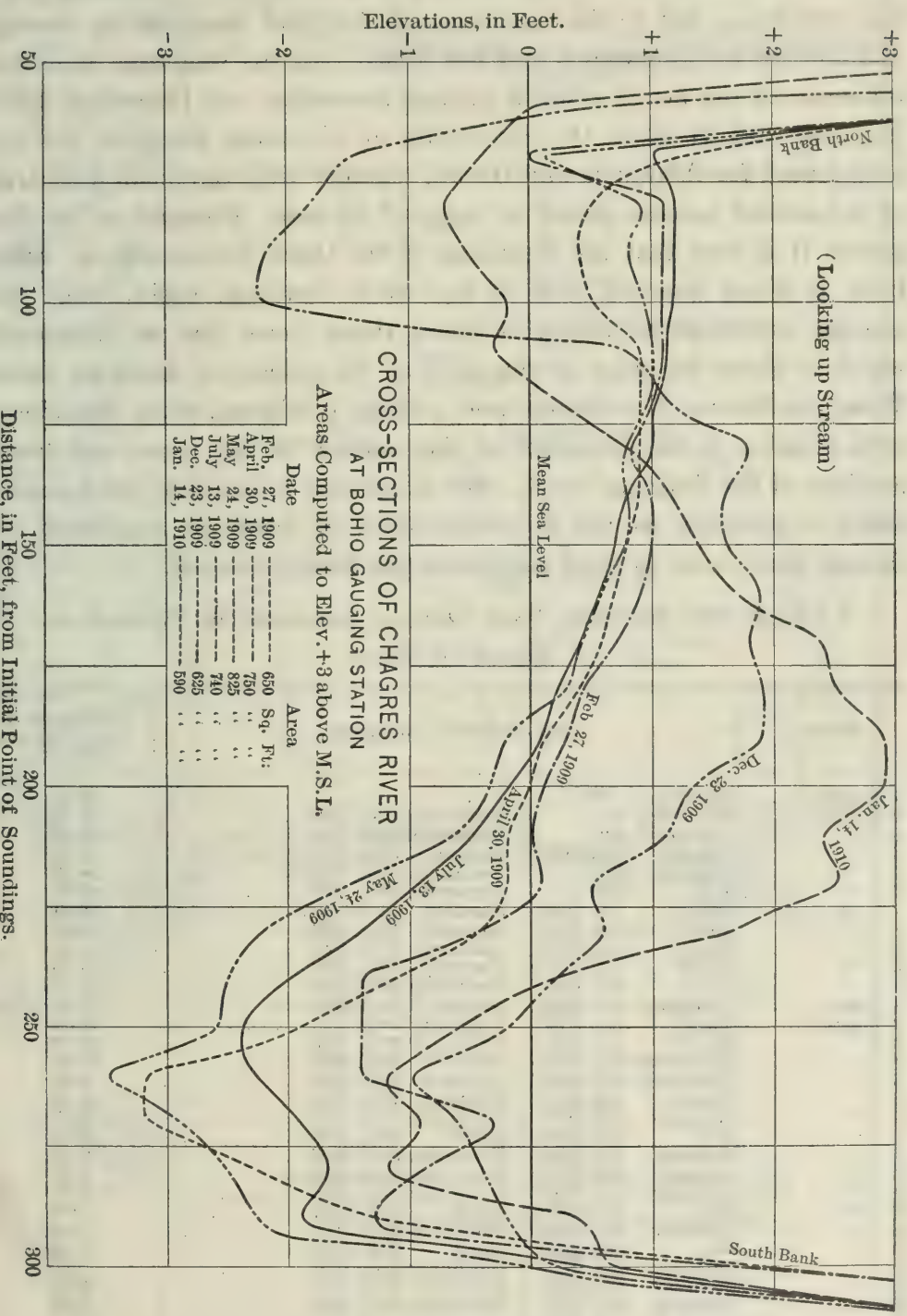
1907	....	....	....	....	2.48	4.90	6.61	4.76	9.32	9.97	7.37	4.72	....
1908	1.25	0.66	0.45	0.32	4.20	3.62	5.58	6.21	5.32	7.74	14.34	4.94	4.55
1909	4.84	3.70	1.24	0.90	3.58	9.21	6.68	6.15	7.33	9.13	19.30	20.46	7.71
1910	6.34	3.47	2.12	4.33	7.48	8.97	11.02	10.23	9.13	10.15	12.60	19.68	8.79
Means (3 and 4 Years).	4.14	2.61	1.27	1.85	4.44	6.68	7.47	6.84	7.78	9.25	13.40	12.45	6.52

Inches on Water-shed.

													Total.
1907	.....	.....	.....	.....	2.859	5.467	7.621	5.488	10.398	11.494	8.223	5.442	.....
1908	1.441	0.712	0.519	0.357	4.842	4.039	6.433	7.159	5.936	8.923	15.998	5.695	62.054
1909	5.580	3.853	1.430	1.004	4.127	10.276	7.701	7.090	8.178	10.526	21.530	23.586	104.881
1910	7.309	3.613	2.444	4.831	8.624	10.008	12.704	11.793	10.186	11.700	14.060	22.688	119.960
Means (3 and 4 Years).	4.777	2.726	1.464	2.064	5.113	7.448	8.615	7.882	8.674	10.661	14.953	14 353	88.730

At Bohio the conditions are very different from those in the upper river. Here the river bed has very little slope, and the material carried along by the flood is alluvium and fine sandy clay. Besides this, the conditions are greatly modified by the large quantity of material from the canal excavation, which is dumped directly into the river and carried down stream. This has resulted in filling the bed to a greater extent than would otherwise have been the case. The changes in cross-section due to the various causes are shown by Fig. 26, the maximum change being about 200 sq. ft. below Elevation + 3 during the period between May 24th and December 23d, 1909. Since April, 1910, back-water conditions, and consequent lessening of the current, have resulted in a comparatively rapid filling of the bed at this point. On account of construction work at Gatun, it has been rather difficult to find a permanent gauging section at this station. Although the conditions of bottom and flow were not ideal, the best location seemed to be in the spillway channel below the concreted portion. Here the bottom was rather rough, and a little back-water was found near shore on each side. The eddies, cross-currents, and waves, which were found farther up in the concreted portion, due to bridge piers and other obstructions, were absent at the site chosen, and this location seemed to have the most advantages. A profile of the bottom, taken on February 28th, 1911, and the horizontal velocity curves taken on February 14th, 21st, and 28th, at different elevations of the lake surface, are shown by Fig. 27.

During the 21 years of record and estimate, for which the discharge tables have been made up, Alhajuela, Gamboa, and Bohio had the maximum discharge year in 1909, and Gatun in 1910. This condition was brought about by the fact that the rainfall and consequent run-off which took place in November and December, 1909, was especially large during the latter part of December, 1909. This caused an excess of flow at the upper stations for the last part of the calendar year, 1909, but much of the water was not delivered at Gatun until the following January, thus appearing to bring up the average of the calendar year, 1910. The river year, 1909-10, however, was the year of maximum run-off at all stations. The year of minimum run-off was 1905 at all stations, and the year of minimum dry-season run-off was 1908. The interrelation of these periods of discharge, as compared with each other and with the mean discharge for the period, is shown by Figs. 29 to 32, inclusive. From these it may be seen that the ratio


Elevations, in Feet.
-3
-2
-1
0
+1
+2
+3
50
100
150
200
250
300
Distance, in Feet, from Initial Point of Soundings.
North Bank
(Looking up Stream)
Mean Sea Level
South Bank
Feb. 27, 1909
April 30, 1909
May 24, 1909
July 13, 1909
Dec. 23, 1909
Jan. 14, 1910
CROSS-SECTIONS OF CHAGRES RIVER
AT BOHIO GAUGING STATION
Areas Computed to Elev. +3 above M.S.L.
Date
Area
Feb. 27, 1909 ---------- 650 Sq. Ft.
April 30, 1909 ---------- 750 " "
May 24, 1909 ---------- 825 " "
July 13, 1909 ---------- 740 " "
Dec. 23, 1909 ---------- 625 " "
Jan. 14, 1910 ---------- 590 " "

of the annual minimum and maximum discharge at Alhajuela is about 3.25, at Gamboa 3.40, at Bohio 2.56, and at Gatun 2.50, decrease down the river being due to the regulation of the flood discharge by flowage and storage in the swamps and low areas. All the diagrams show the influence of the heavy rainfall during November and December, 1909. Figs. 33 and 34 show the conditions of discharge duration for the period used for Alhajuela and Gatun, together with the mean discharge of the several months placed in order of dryness. Comparing the diagrams, it is seen that the discharge of the three dry months at Alhajuela is about one-half that of the whole drainage basin, while the average discharge at Gatun is about three times that at Alhajuela, which is about the same as the ratio of the respective drainage areas. These conditions of minimum and average discharge, when considered with relation to the character of the country in the upper and lower portions of the drainage basin, offer additional evidence of the harmful effect of swamps, as far as conservation of water is concerned, although their value as flood regulators has been indicated.

TABLE 33.—Run-off Past Gatun, Arranged in Periods of Maximum Flow.

Period.	Date of Period (inclusive).	Discharge, in cubic feet per second.
1 Month	December, 1893.....	30 037
2 Months.....	November 1st, to December 31st, 1909.....	27 135
3 "	October 1st, " December 31st, 1909.....	22 287
4 "	October 1st, 1909, " January 31st, 1910.....	19 650
5 "	August 1st, " December 31st, 1893.....	18 470
6 "	July 1st, " December 31st, 1893.....	17 460
7 "	July 1st, 1893, " January 31st, 1894.....	16 600
8 "	June 1st, 1893, " January 31st, 1894.....	15 520
9 "	May 1st, 1893, " January 31st, 1894.....	14 650
10 "	April 1st, 1893, " January 31st, 1894.....	13 740
11 "	April 1st, 1893, " February 28th, 1894.....	12 743
1 Year.....	February 1st, 1893, " January 31st, 1894.....	11 948
2 Years.....	March 1st, 1892, " February 28th, 1894.....	11 366
3 "	March 1st, 1892, " February 28th, 1895.....	10 736
4 "	November 1st, 1891, " October 31st, 1895.....	10 003
5 "	February 1st, 1890, " January 31st, 1895.....	9 890
6 "	March 1st, 1890, " February 29th, 1896.....	9 460
7 "	January 1st, 1890, " December 31st, 1896.....	9 070
8 "	May 1st, 1890, " April 30th, 1898.....	8 951
9 "	January 1st, 1890, " December 31st, 1898.....	8 632
10 "	January 1st, 1890, " December 31st, 1899.....	8 318
11 "	January 1st, 1890, " December 31st, 1900.....	8 080
12 "	May 1st, 1890, " April 30th, 1902.....	8 020
13 "	January 1st, 1890, " December 31st, 1902.....	7 840
14 "	May 1st, 1890, " April 30th, 1904.....	7 780
15 "	January 1st, 1890, " December 31st, 1904.....	7 680
16 "	January 1st, 1890, " December 31st, 1905.....	7 485
17 "	January 1st, 1890, " December 31st, 1906.....	7 430
18 "	January 1st, 1890, " December 31st, 1907.....	7 340
19 "	January 1st, 1892, " December 31st, 1910.....	7 481
20 "	January 1st, 1891, " December 31st, 1910.....	7 472
21 "	January 1st, 1890, " December 31st, 1910.....	7 602

PLATE XXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1260.
SAVILLE ON
HYDROLOGY OF THE
PANAMA CANAL.

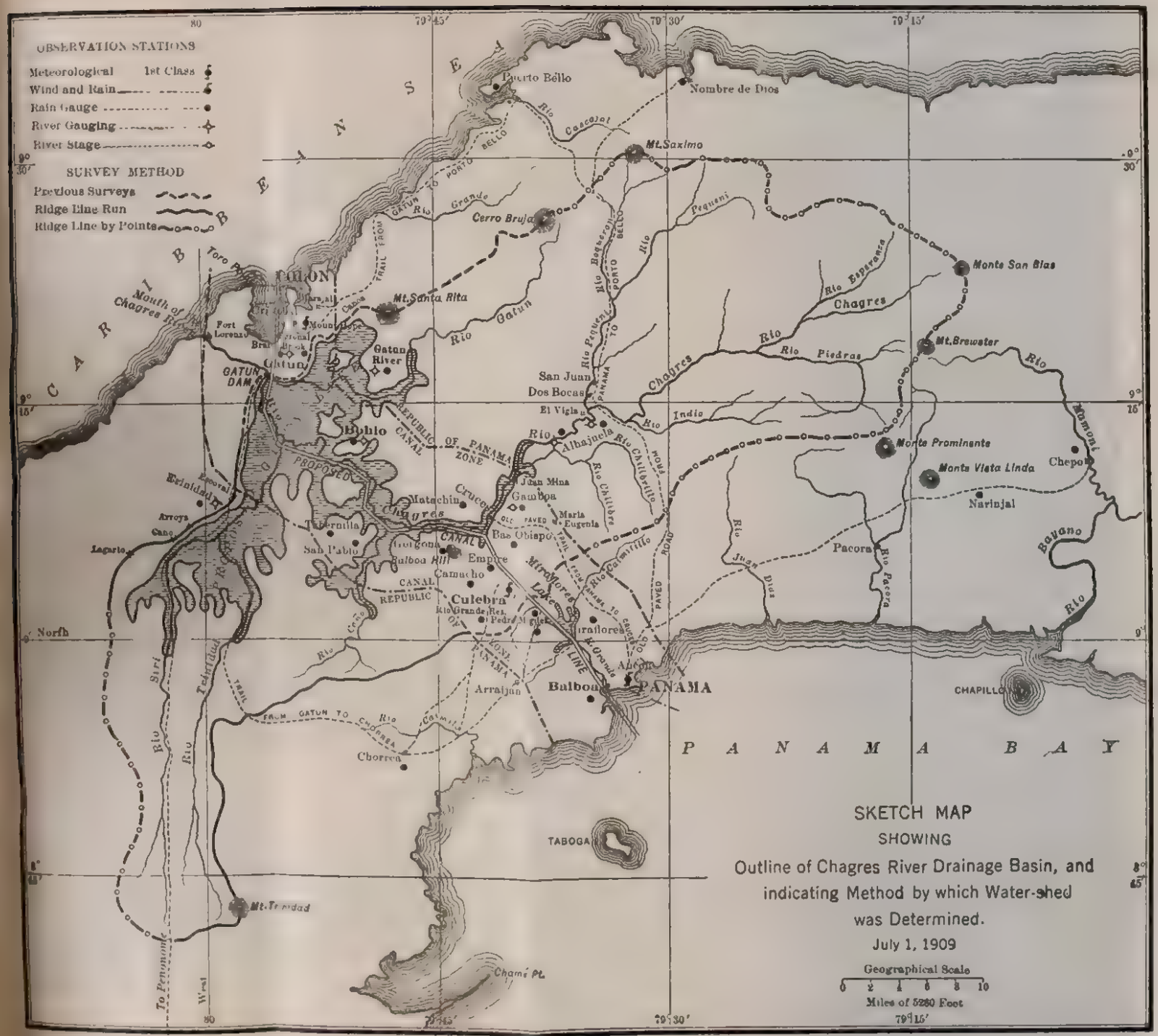


TABLE 34.—Run-off Past Gatun, Arranged in Periods of Minimum Flow.

Period.	Date of Period (inclusive).	Discharge, in cubic feet per second.
1 Month	March, 1908	851
2 Months	March 1st, to April 30th, 1908	872
3 "	February 1st, " April 30th, 1908	930
4 "	January 1st, " April 30th, 1908	1 191
5 "	January 1st, " May 31st, 1903	1 739
6 "	January 1st, " June 30th, 1903	2 136
7 "	December 1st, 1902, " June 30th, 1903	2 526
8 "	December 1st, 1902, " July 31st, 1903	3 025
9 "	January 1st, 1905, " September 30th, 1905	3 486
10 "	December 1st, 1904, " September 30th, 1905	3 814
11 "	July 1st, 1905, " May 31st, 1906	4 449
1 Year	June 1st, 1905, " May 31st, 1906	4 435
2 Years	September 1st, 1899, " August 31st, 1901	5 163
3 "	September 1st, 1898, " August 31st, 1901	5 310
4 "	August 1st, 1904, " July 31st, 1908	5 544
5 "	August 1st, 1898, " July 31st, 1903	5 815
6 "	July 1st, 1902, " June 30th, 1908	5 823
7 "	July 1st, 1899, " June 30th, 1906	5 928
8 "	August 1st, 1898, " July 31st, 1906	5 902
9 "	May 1st, 1899, " April 30th, 1908	5 972
10 "	August 1st, 1898, " July 31st, 1908	5 937
11 "	February 1st, 1898, " January 31st, 1909	6 032
12 "	July 1st, 1896, " June 30th, 1908	6 255
13 "	November 1st, 1895, " October 31st, 1908	6 239
14 "	February 1st, 1895, " January 31st, 1909	6 314
15 "	February 1st, 1894, " January 31st, 1909	6 749
16 "	November 1st, 1893, " October 31st, 1909	6 769
17 "	November 1st, 1892, " October 31st, 1909	6 970
18 "	January 1st, 1891, " December 31st, 1908	7 092
19 "	November 1st, 1890, " October 31st, 1909	7 138
20 "	November 1st, 1890, " October 31st, 1910	7 411
21 "	January 1st, 1890, " December 31st, 1910	7 602

The critical period in maximum discharge appears to be a space of about 1 month in the case of Gatun; that is, a fairly uniformly increasing discharge can be expected for about 11 months of the year, with the rate in the twelfth month greatly in excess.

At Alhajuela this period of excess flow is of shorter duration, being only about 10 months out of the 252 recorded, or an average of about 15 days per year.

Fig. 35 shows the interrelations of discharge at the four main stations on the Chagres River.

Figs. 36, 37, and 38 show the cycle of monthly average discharge, or the average hydrograph, for the years of record. From these and the preceding diagrams the probable quantity of water available for power is easily seen. For the design of spillway and discharge channels, absolute maximum conditions are required, and these may be found in Table 35.

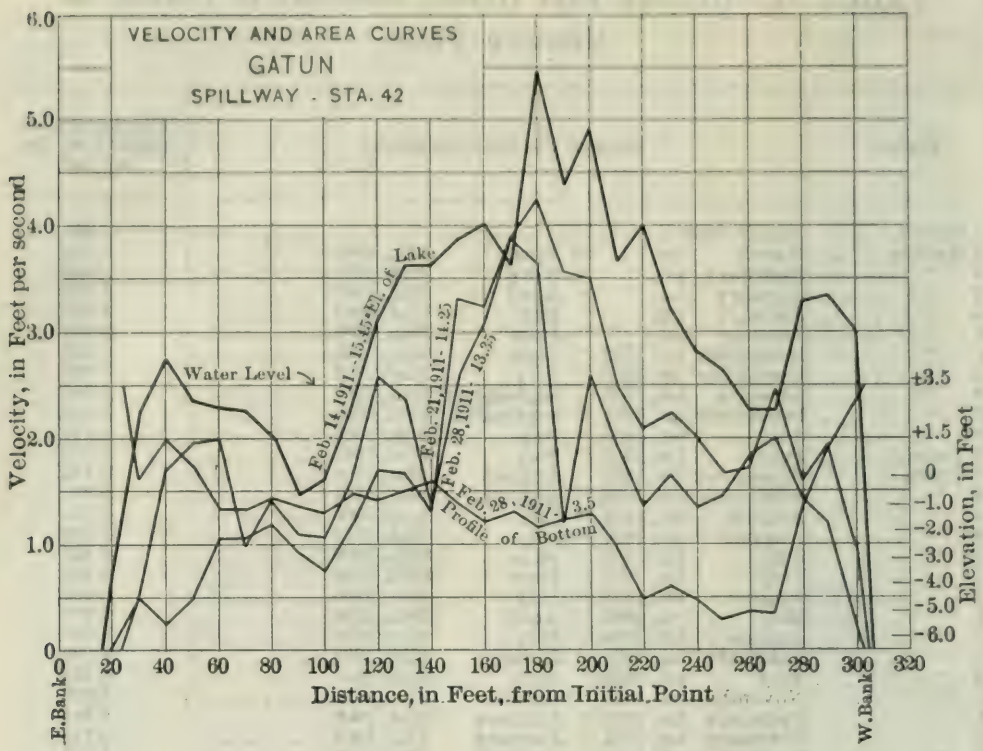

VELOCITY AND AREA CURVES
GATUN
SPILLWAY - STA. 42
Velocity, in Feet per second
Elevation, in Feet
Distance, in Feet, from Initial Point
E. Bank
W. Bank
Water Level
Lake
Feb. 14, 1911 - 15.45 El. of
Feb. 21, 1911 - 14.25
Feb. 28, 1911 - 13.25
Feb. 28, 1911 - 3.5
Profile of Bottom

FIG. 27.

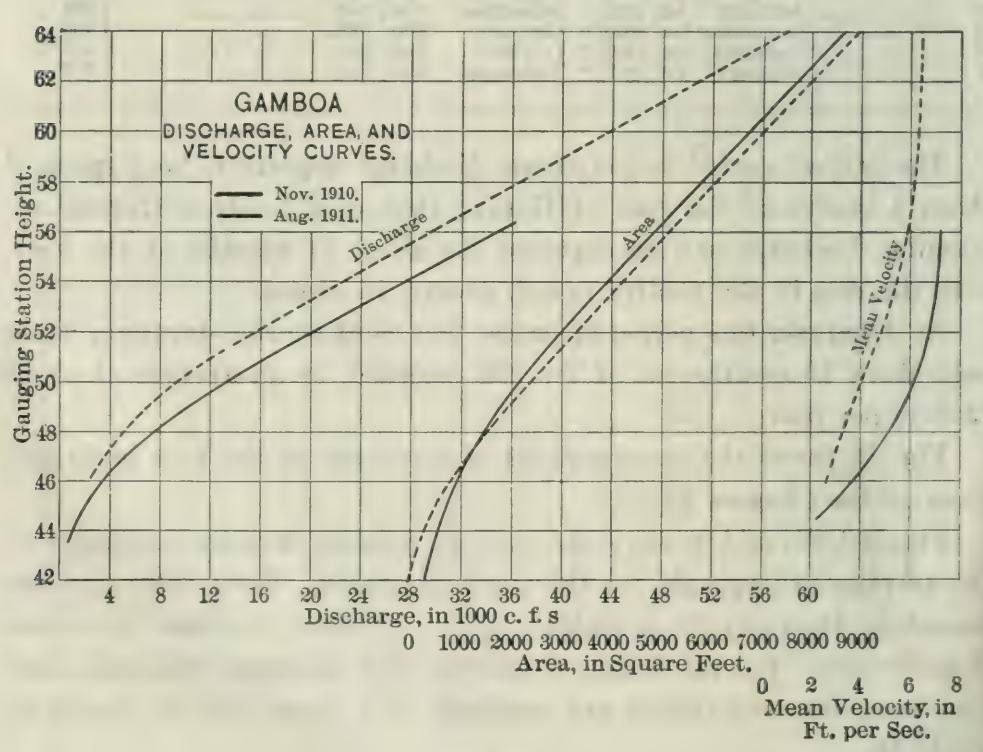

GAMBOA
DISCHARGE, AREA, AND
VELOCITY CURVES.
Nov. 1910.
Aug. 1911.
Gauging Station Height.
Discharge
Area
Mean Velocity
Discharge, in 1000 c. f. s
Area, in Square Feet.
Mean Velocity, in
Ft. per Sec.

FIG. 28.

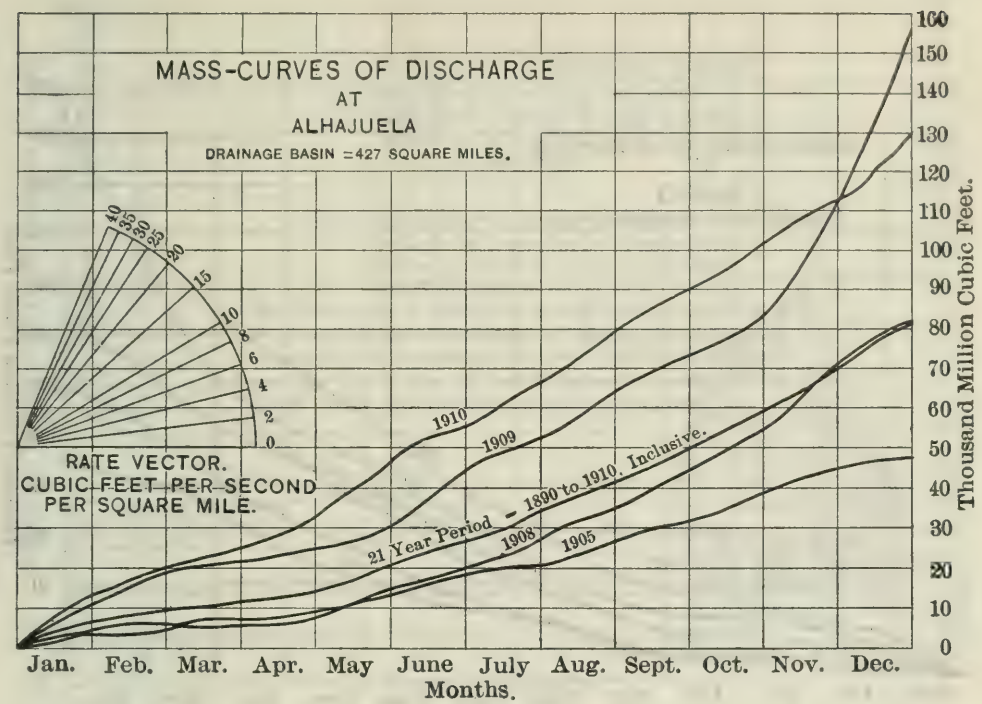

MASS-CURVES OF DISCHARGE
AT
ALHAJUELA
DRAINAGE BASIN =427 SQUARE MILES.
RATE VECTOR.
CUBIC FEET PER SECOND
PER SQUARE MILE.
40
35
30
25
20
15
10
8
6
4
2
0
1910
1909
21 Year Period - 1890 to 1910, Inclusive.
1908
1905
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
Thousand Million Cubic Feet.
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
Months.

FIG. 29.

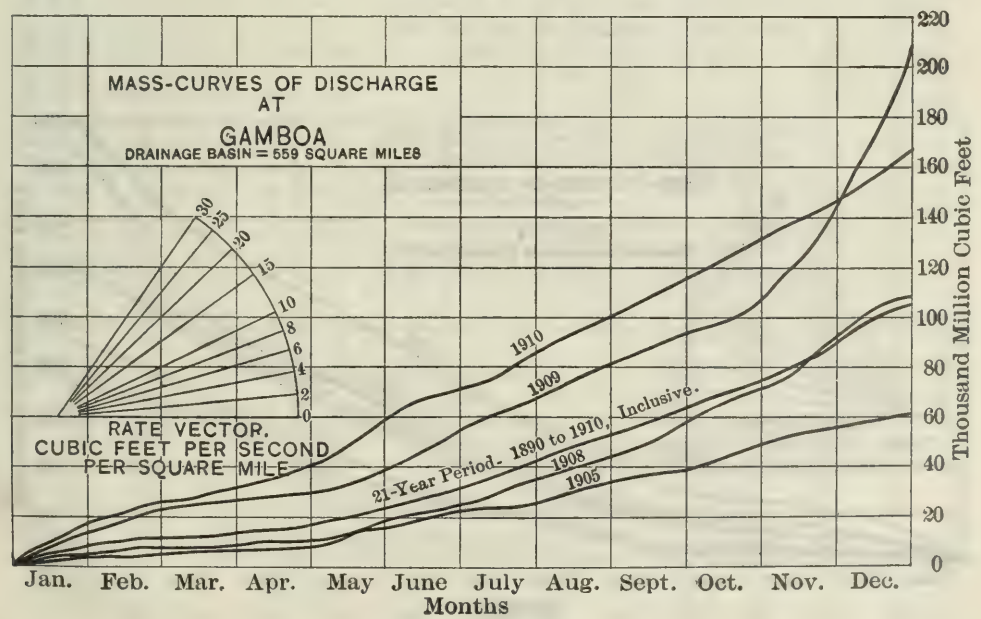

MASS-CURVES OF DISCHARGE
AT
GAMBOA
DRAINAGE BASIN = 559 SQUARE MILES
RATE VECTOR.
CUBIC FEET PER SECOND
PER SQUARE MILE
30
25
20
15
10
8
6
4
2
0
1910
1909
21-Year Period- 1890 to 1910, Inclusive.
1908
1905
220
200
180
160
140
120
100
80
60
40
20
0
Thousand Million Cubic Feet
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
Months

FIG. 30.

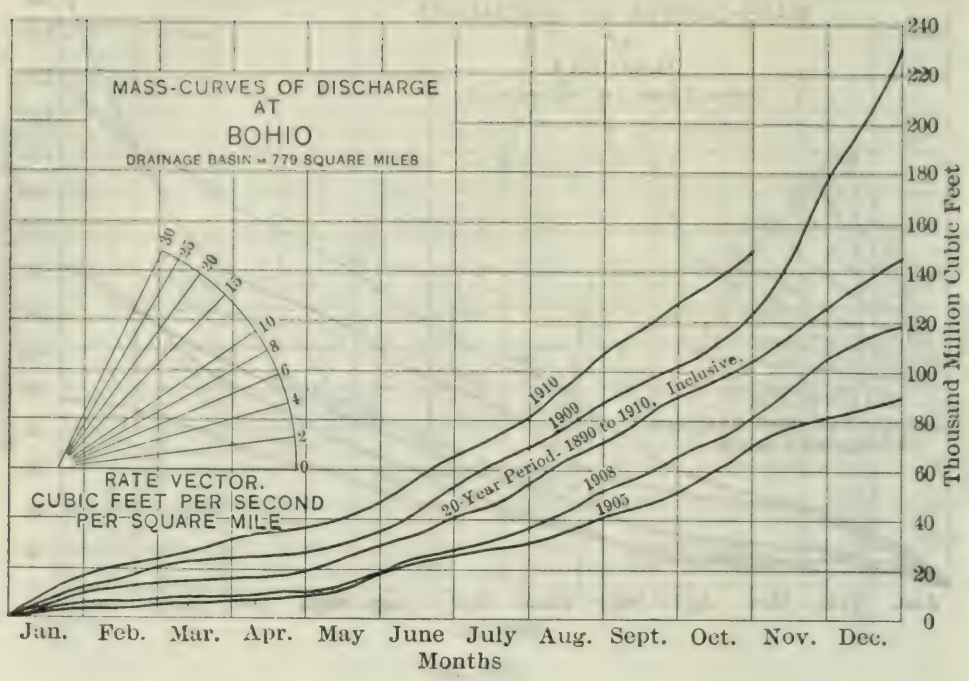

MASS-CURVES OF DISCHARGE
AT
BOHIO
DRAINAGE BASIN = 779 SQUARE MILES
RATE VECTOR.
CUBIC FEET PER SECOND
PER SQUARE MILE
25
20
15
10
8
6
4
2
0
1910
1909
20-Year Period- 1890 to 1910, Inclusive.
1908
1905
240
220
200
180
160
140
120
100
80
60
40
20
0
Thousand Million Cubic Feet
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
Months

FIG. 31.

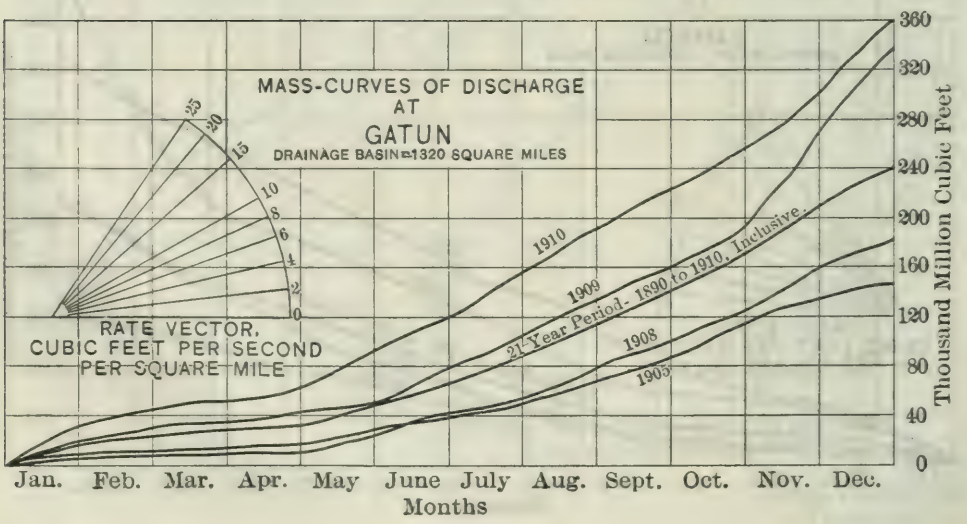

MASS-CURVES OF DISCHARGE
AT
GATUN
DRAINAGE BASIN=1320 SQUARE MILES
RATE VECTOR.
CUBIC FEET PER SECOND
PER SQUARE MILE
25
20
15
10
8
6
4
2
0
1910
1909
21-Year Period- 1890 to 1910, Inclusive.
1908
1905
360
320
280
240
200
160
120
80
40
0
Thousand Million Cubic Feet
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
Months

FIG. 32.

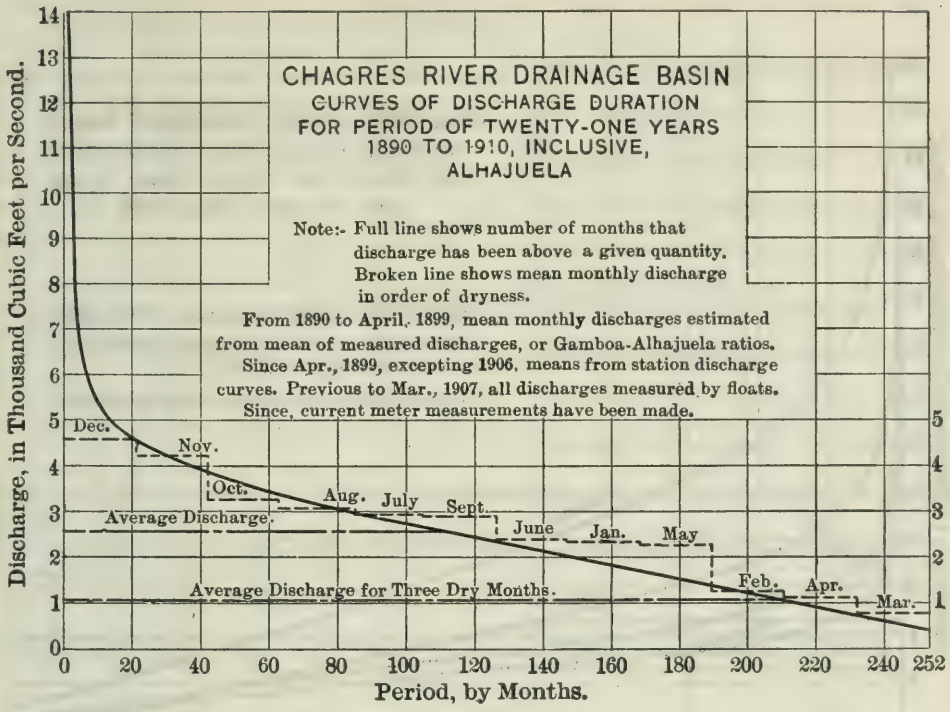

CHAGRES RIVER DRAINAGE BASIN
CURVES OF DISCHARGE DURATION
FOR PERIOD OF TWENTY-ONE YEARS
1890 TO 1910, INCLUSIVE,
ALHAJUELA
Note:- Full line shows number of months that discharge has been above a given quantity.
Broken line shows mean monthly discharge in order of dryness.
From 1890 to April, 1899, mean monthly discharges estimated from mean of measured discharges, or Gamboa-Alhajuela ratios.
Since Apr., 1899, excepting 1906, means from station discharge curves. Previous to Mar., 1907, all discharges measured by floats.
Since, current meter measurements have been made.
Dec.
Nov.
Oct.
Aug.
July
Sept.
June
Jan.
May
Feb.
Apr.
Mar.
Average Discharge.
Average Discharge for Three Dry Months.
Discharge, in Thousand Cubic Feet per Second.
Period, by Months.

FIG. 33.

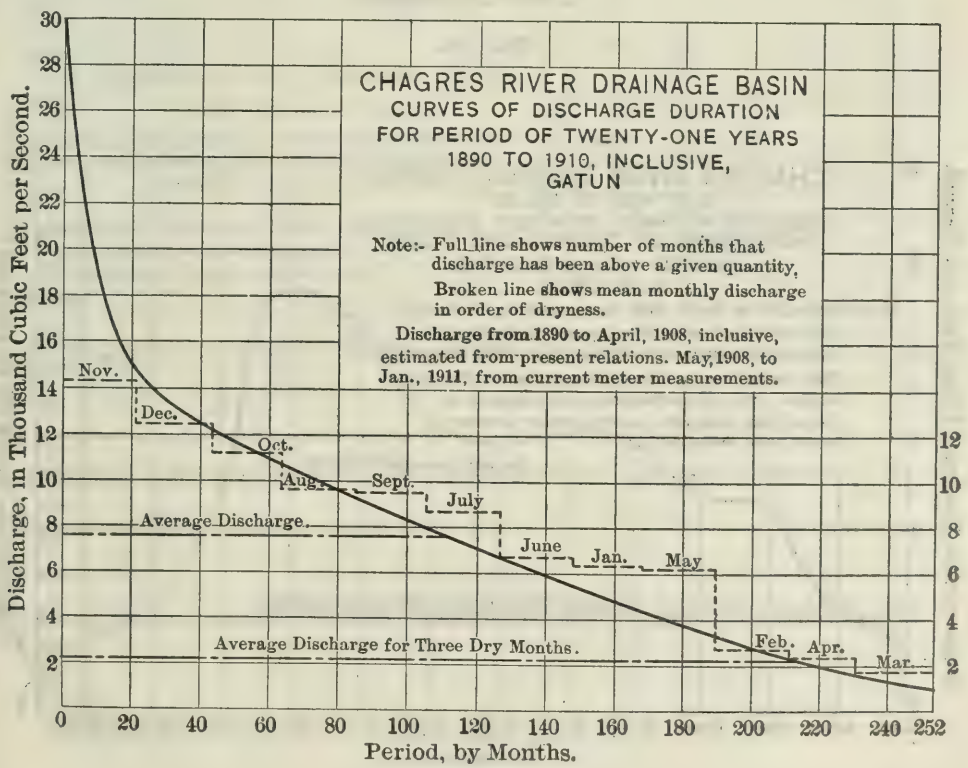

CHAGRES RIVER DRAINAGE BASIN
CURVES OF DISCHARGE DURATION
FOR PERIOD OF TWENTY-ONE YEARS
1890 TO 1910, INCLUSIVE,
GATUN
Note:- Full line shows number of months that discharge has been above a given quantity,
Broken line shows mean monthly discharge in order of dryness.
Discharge from 1890 to April, 1908, inclusive, estimated from present relations. May, 1908, to Jan., 1911, from current meter measurements.
Nov.
Dec.
Oct.
Aug.
Sept.
July
June
Jan.
May
Feb.
Apr.
Mar.
Average Discharge.
Average Discharge for Three Dry Months.
Discharge, in Thousand Cubic Feet per Second.
Period, by Months.

FIG. 34.

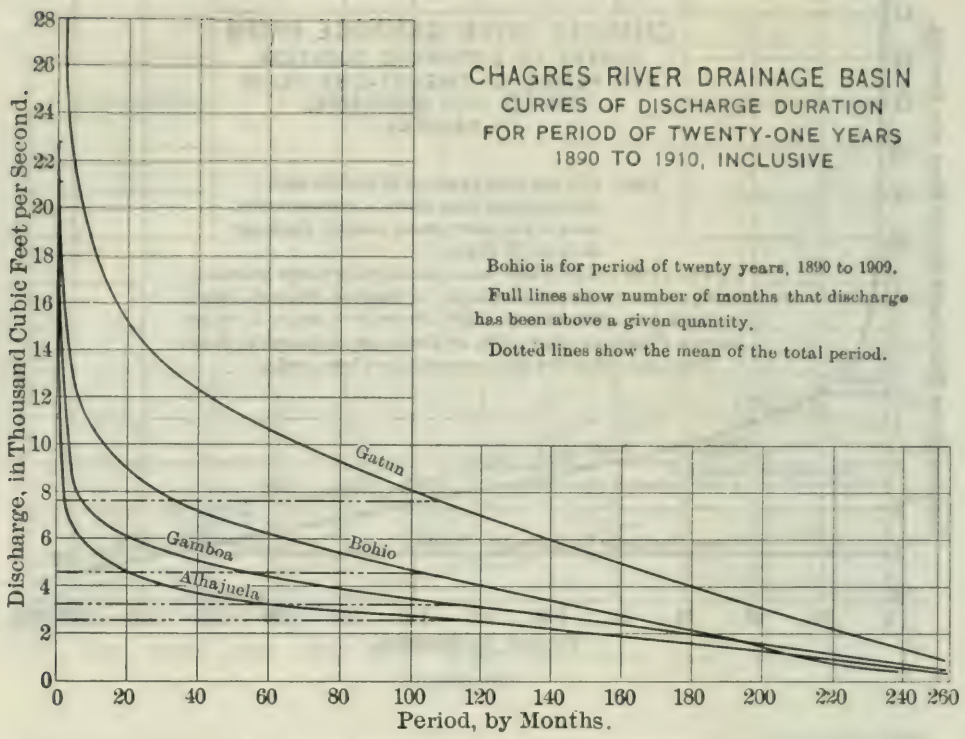

CHAGRES RIVER DRAINAGE BASIN
CURVES OF DISCHARGE DURATION
FOR PERIOD OF TWENTY-ONE YEARS
1890 TO 1910, INCLUSIVE
Bohio is for period of twenty years, 1890 to 1909.
Full lines show number of months that discharge has been above a given quantity.
Dotted lines show the mean of the total period.
Gatun
Bohio
Gamboa
Alhajuela
Discharge, in Thousand Cubic Feet per Second.
Period, by Months.
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28
0 20 40 60 80 100 120 140 160 180 200 220 240 260

FIG. 35.

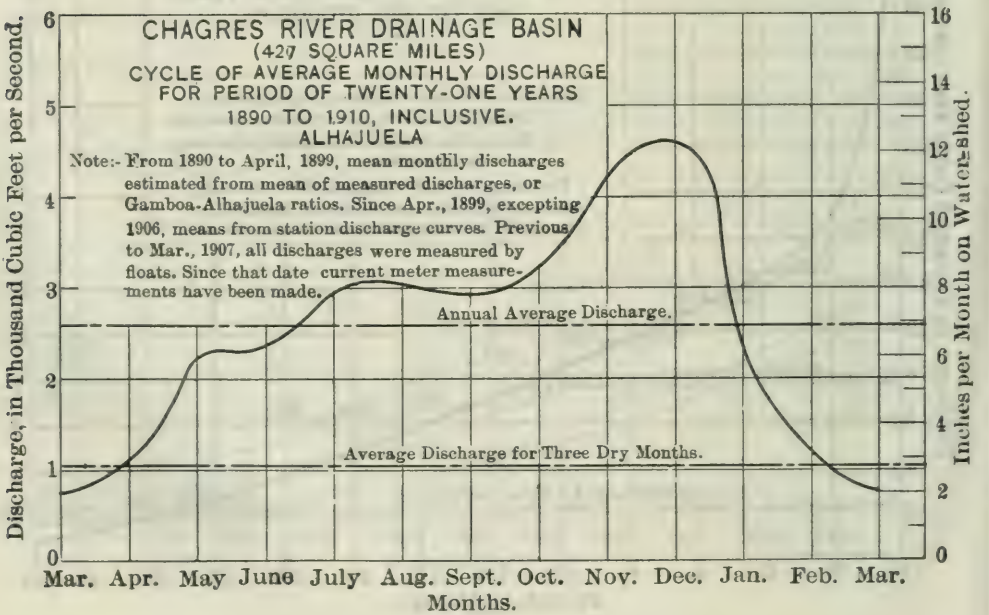

CHAGRES RIVER DRAINAGE BASIN
(427 SQUARE MILES)
CYCLE OF AVERAGE MONTHLY DISCHARGE
FOR PERIOD OF TWENTY-ONE YEARS
1890 TO 1910, INCLUSIVE.
ALHAJUELA
Note:- From 1890 to April, 1899, mean monthly discharges estimated from mean of measured discharges, or Gamboa-Alhajuela ratios. Since Apr., 1899, excepting 1906, means from station discharge curves. Previous to Mar., 1907, all discharges were measured by floats. Since that date current meter measurements have been made.
Annual Average Discharge.
Average Discharge for Three Dry Months.
Discharge, in Thousand Cubic Feet per Second.
Inches per Month on Watershed.
Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar.
Months.

FIG. 36.

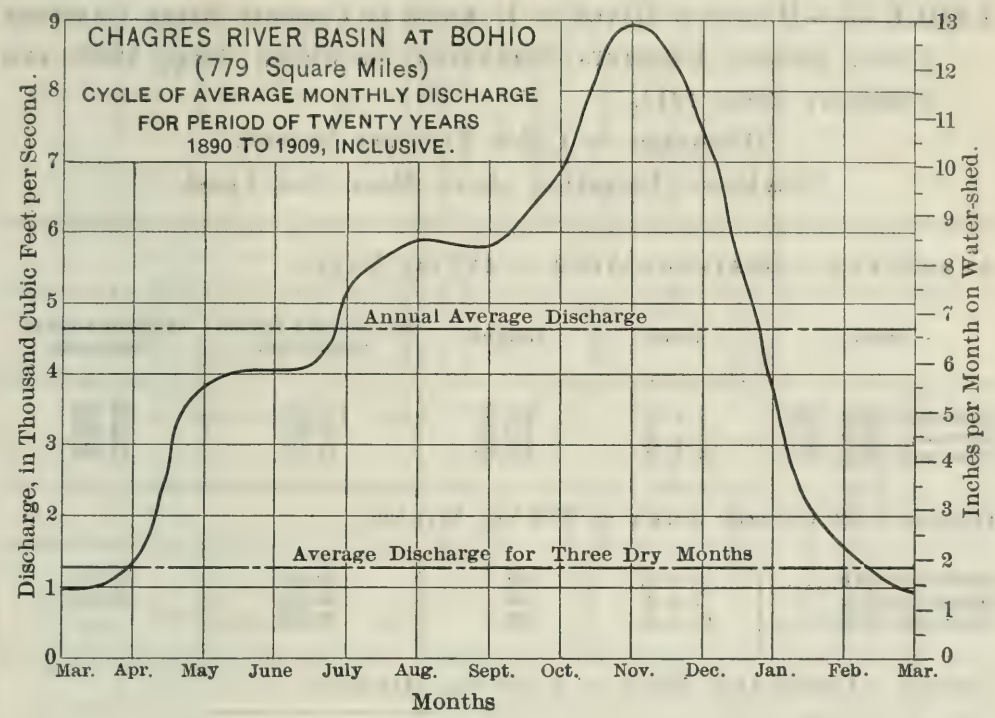

CHAGRES RIVER BASIN AT BOHIO
(779 Square Miles)
CYCLE OF AVERAGE MONTHLY DISCHARGE
FOR PERIOD OF TWENTY YEARS
1890 TO 1909, INCLUSIVE.
Annual Average Discharge
Average Discharge for Three Dry Months
Discharge, in Thousand Cubic Feet per Second.
Inches per Month on Water-shed.
Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar.
Months

FIG. 37.

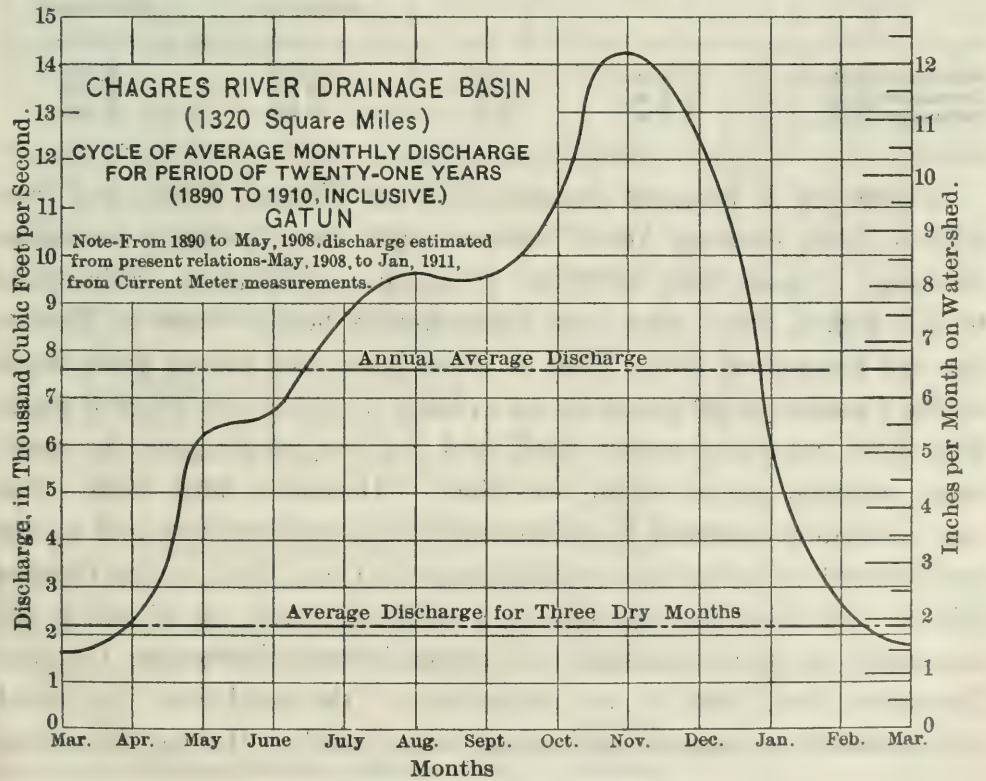

CHAGRES RIVER DRAINAGE BASIN
(1320 Square Miles)
CYCLE OF AVERAGE MONTHLY DISCHARGE
FOR PERIOD OF TWENTY-ONE YEARS
(1890 TO 1910, INCLUSIVE.)
GATUN
Note-From 1890 to May, 1908, discharge estimated
from present relations-May, 1908, to Jan, 1911,
from Current Meter measurements.
Annual Average Discharge
Average Discharge for Three Dry Months
Discharge, in Thousand Cubic Feet per Second.
Inches per Month on Water-shed.
Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar.
Months

FIG. 38.

TABLE 35.—Maximum Rates of Run-off in Chagres River Drainage Basin, During Freshets Beginning December 26th, 1909, and February 12th, 1911.

Discharge = Cubic Feet per Second.
Height = Elevation above Mean Sea Level.

Alhajuela.—Drainage Area = 427 Sq. Miles.

Date.	Time.	Height.	Momentary rate of discharge.	24-Hour rate of discharge.
December 26th, 1909..	7 P. M.	121.00	170 000	95 700
December 30th, 1909..	11 A. M.	112.00	97 200	43 500
February 12th, 1911..	11 P. M.	105.50	43 600	17 280

Bohio.—Drainage Area = 779 Sq. Miles.

December 27th.....	11 A. M.	38.7	90 000	78 400
December 31st.....	2 A. M.	32.1	58 500	38 600
February 13th.....	10 A. M.	21.1	37 500	.....

Gatun.—Drainage Area = 1 320 Sq. Miles.

Date.	Time.	Height.	Momentary rate of run-off (storage + discharge).	24-Hour rate of run-off (storage + discharge).
December 28th.....	9 A. M.	19.65	124 000	94 000
December 31st.....	9 A. M.	17.7	60 000	57 000
February 13th.....	4 P. M.	15.7	38 860	21 520

Floods are of frequent occurrence in the Chagres Basin, and from 1879 to 1906, General Abbot* records eight rises having a maximum discharge of more than 40 000 cu. ft. per sec. at Gamboa. Subsequent to this period, there have been other similar floods—those of November and December, 1909. That is, there have been eleven great floods in the Chagres in 30 years, or an average of about one every 3 years. The most important recent flood, and the one which gave the maximum recorded gauge height, was that of December 26th, 1909. This was accurately observed in all its conditions, and the data and results are believed to be the most reliable record of any flood on the Chagres River. The flood was the last of a series of three, all of which can be classed as great, and their occurrence between November 11th and December 31st, 1909, is an indication of the conditions for which it is necessary to provide in tropical water supply. During the period,

* "Problems of the Panama Canal," p. 155.

TABLE 36.—Comparative Rainfall During Flood Periods.

December 26th, 27th, 28th, 29th, and 30th, 1909.
November 16th, 17th, 18th, 19th, and 20th, 1909.
December 1st, 2d, 3d, 4th, and 5th, 1906.

	1st Period.			2d Period.			3d Period.			4th Period.			5th Period.			Total. Dec., 1906, 26th to 30th, inclusive.	Total. Nov., 1909, 16th to 20th, inclusive.	Total. Dec., 1906, 1st to 5th, inclusive.
December, 1909.	26th.			27th.			28th.			29th.			30th.					
November, 1909.		16th.			17th.			18th.			19th.			20th.				
December, 1906.			1st.			2d.			3d.			4th.			5th.			
Nombre de Dios	3.43	0.81	.....	2.38	7.08	.....	1.79	6.64	.....	0.57	4.49	.....	3.15	0.48	.....	11.32	19.50	.....
Porto Bello	5.55	3.36	.....	2.21	1.45	.....	3.64	8.53	.....	7.96	2.84	.....	4.94	0.28	.....	24.30	16.46	.....
Cristobal	3.69	0.34	0.04	1.85	3.40	0.13	0.50	3.39	8.47	0.36	0.73	0.39	3.29	1.19	0.05	9.69	9.05	9.08
Brazos Brook	4.05	0.31	0.00	1.43	3.82	0.10	1.13	3.41	8.96	0.43	1.46	0.45	3.33	0.78	0.00	10.37	9.78	9.51
Gatun	3.99	1.19	0.02	3.84	3.64	0.25	0.21	3.74	10.48	0.30	1.63	0.44	2.36	0.56	0.06	10.70	10.76	11.25
Monte Lirio	5.35	0.66	.....	1.00	3.83	.....	0.62	3.18	.....	0.10	1.55	.....	2.77	0.45	.....	9.84	9.67	.....
Lagartera	3.10	0.60	.....	0.51	4.51	.....	0.61	2.98	.....	0.18	1.42	.....	2.35	0.43	.....	6.75	9.84	.....
Bohio	3.77	2.26	0.21	0.54	2.15	0.34	0.55	2.96	6.13	0.51	0.99	0.15	2.23	0.36	0.05	7.60	8.72	6.88
Empire	1.67	0.52	2.63	1.71	2.59	0.48	0.22	1.11	6.15	0.09	0.73	0.02	1.87	0.05	0.06	5.56	5.01	9.34
Camacho	1.32	1.10	2.21	2.11	3.20	0.15	0.41	1.10	5.53	0.07	1.46	0.10	2.61	0.08	0.00	6.52	6.94	7.99
Culebra	1.51	0.90	1.97	1.72	3.17	0.34	0.41	1.56	5.55	0.11	1.08	0.01	2.47	0.07	0.04	6.22	6.78	7.91
Gamboa	2.66	0.75	1.61	1.10	1.96	0.36	0.32	2.18	6.29	0.34	1.57	0.04	2.18	0.12	0.00	6.60	6.58	8.30
Alhajuela	5.56	0.31	0.66	1.70	3.76	0.28	3.49	3.62	8.19	1.09	2.22	0.14	2.91	0.54	0.00	14.75	10.45	9.27
Rio Grande	1.70	0.74	2.63	1.74	3.33	0.47	0.20	1.33	5.58	0.11	0.90	0.00	2.11	0.08	0.02	5.86	6.38	8.70
Balboa	2.71	0.19	0.50	1.28	1.67	0.21	0.01	1.75	2.40	0.25	0.32	0.20	1.30	0.06	0.02	5.55	3.99	3.33
Ancon	2.16	0.10	0.42	1.31	1.61	1.61	0.06	1.64	1.58	0.20	0.31	0.22	1.16	0.22	0.00	4.89	3.88	3.83

it is estimated that 128 000 000 000 cu. ft. of water were discharged past Gatun. This quantity, if delivered under the present conditions of Gatun Lake (water surface + 15) and with no outlet, would fill the lake to about Elevation + 73, or to within 14 ft. of maximum lake level (+ 87). Hydrographs of two of these floods and mass-curves of discharge are shown on Plates XXIX and XXX. These hydro-

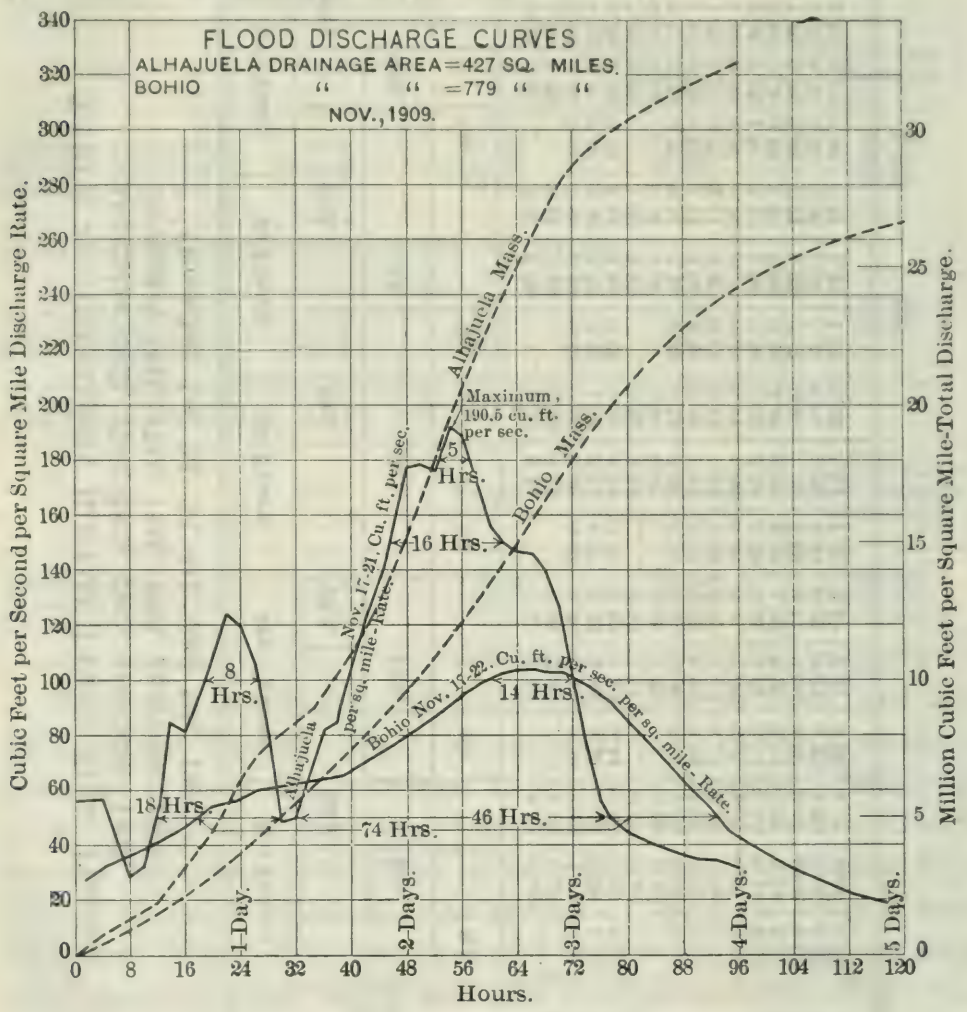


FIG. 39.

graphs are typical of the Chagres River freshets, and a study of them presents some very interesting facts. Figs. 39 and 40 show these conditions, and indicate two distinct types of freshet discharge which are probably of not infrequent occurrence in a long term of years, although they have been unusual during the period in which authentic records are available. The freshet of November, 1909, was of com-

946A.

PLATE XXIX.
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HYDROLOGY OF THE
PANAMA CANAL.

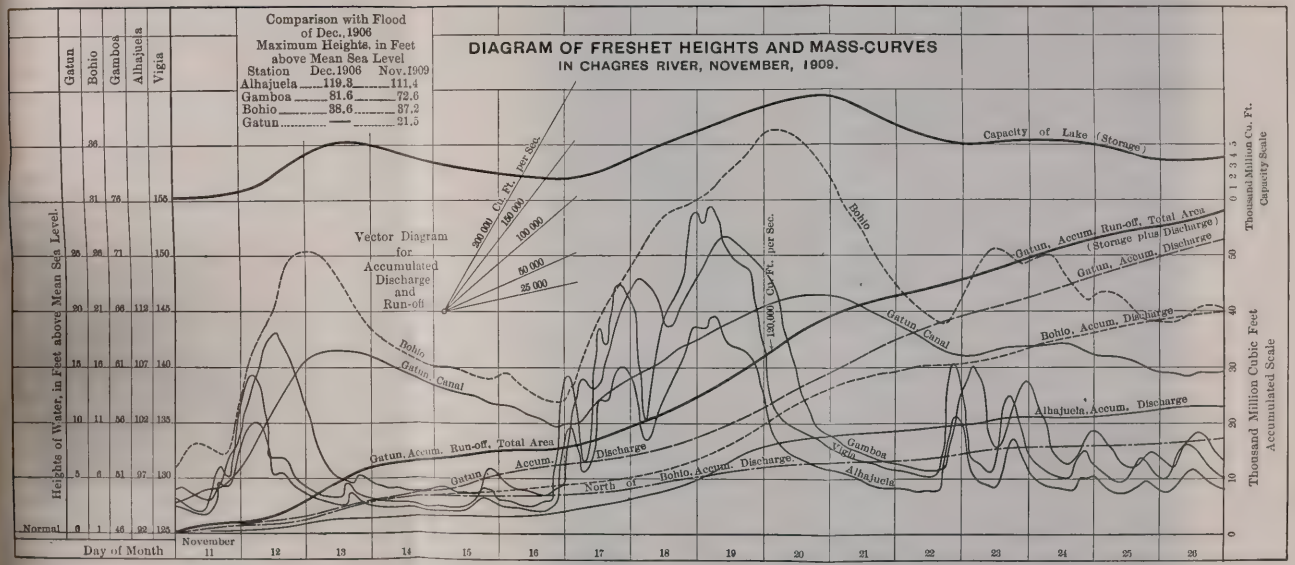
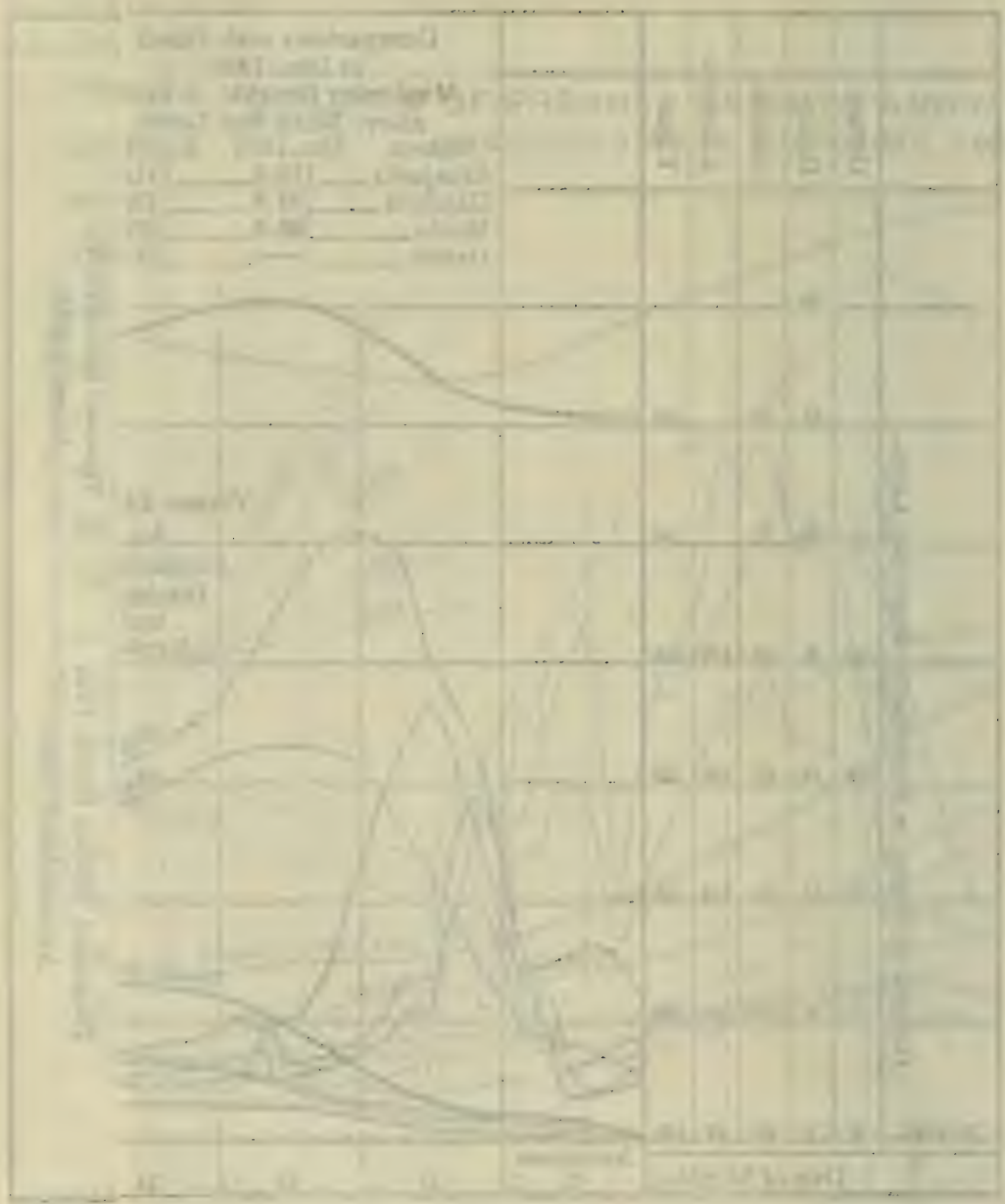


DIAGRAM OF FRESHET HEIGHTS AND MASS-CURVES
IN CHAGRES RIVER, NOVEMBER, 1909.
Comparison with Flood of Dec., 1906
Maximum Heights, in Feet above Mean Sea Level
Station Dec. 1906 Nov. 1909
Alhajuela 119.3 111.4
Gamboa 81.6 72.6
Bohio 38.6 37.2
Gatun — 21.5
Gatun Bohio Gamboa Alhajuela Vigia
Heights of Water, in Feet above Mean Sea Level.
Normal 0 1 46 92 135
5 6 51 97 130
10 11 56 102 135
15 16 61 107 140
20 21 66 113 145
25 26 71 150
31 76 155
36
Vector Diagram for Accumulated Discharge and Run-off
200 000 Cu. Ft. per Sec.
150 000
100 000
50 000
25 000
Capacity of Lake (Storage)
Bohio
Gatun Canal
Gatun, Accum. Run-off, Total Area
Gatun, Accum. Discharge
Bohio, Accum. Discharge
Alhajuela, Accum. Discharge
Gamboa
Vigia
Alhajuela
North of Bohio, Accum. Discharge
Gatun, Accum. Run-off, Total Area (Storage plus Discharge)
Thousand Million Cu. Ft. Capacity Scale
Thousand Million Cubic Feet Accumulated Scale
Day of Month
November
11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26



paratively longer duration and of more widely distributed rainfall than that of the succeeding month, although the crest height at all stations was less than that of the December freshet. For the November rise, however, the discharge past Gatun was nearly 50% more than that of

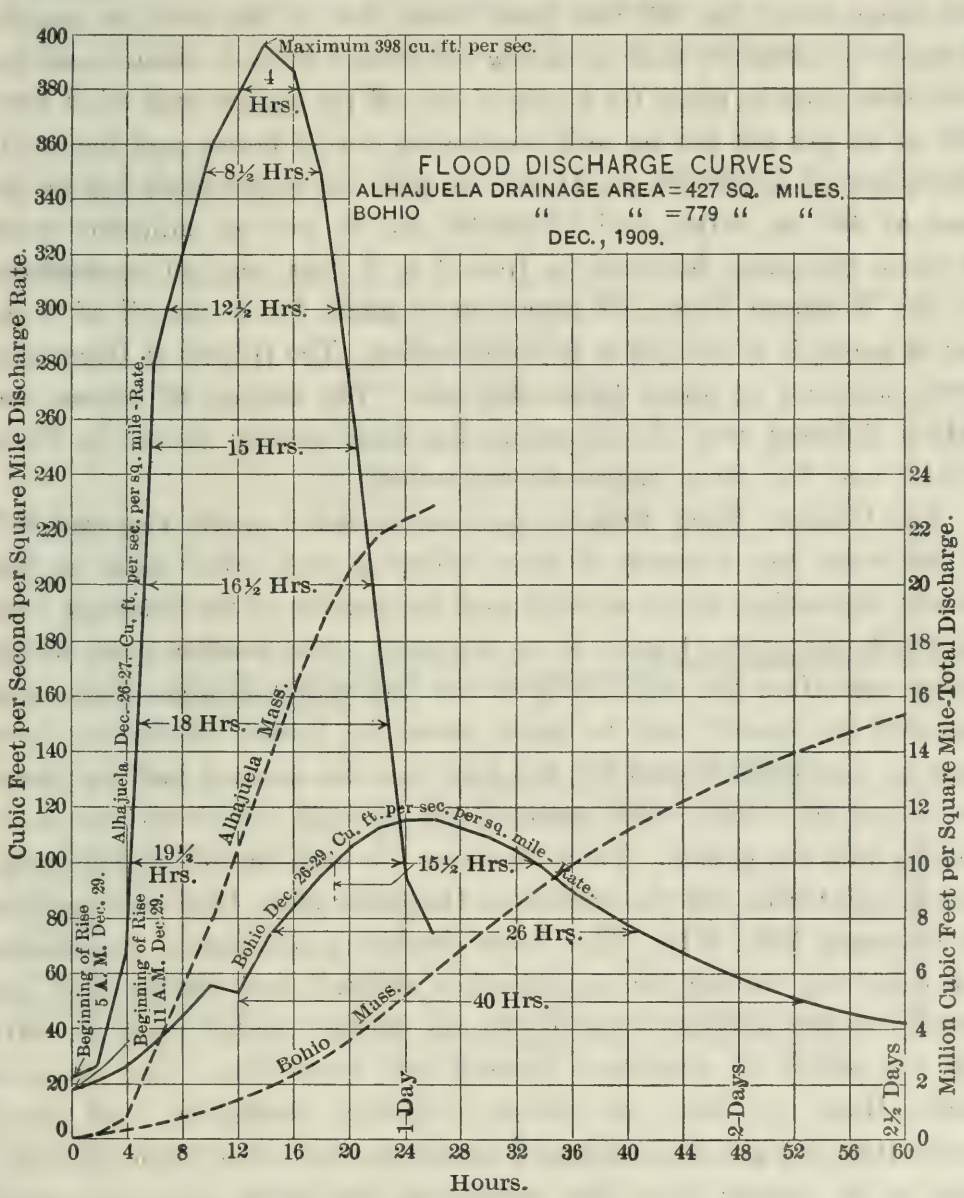


FIG. 40.

December. The duration of those extensive flows is a matter of considerable interest, and the length of time the rate of run-off equalled or exceeded a given rate is shown by the figures on the horizontal lines included between the discharge lines.

On comparing these two curves on this basis, it is seen that in November the flow was more than 100 cu. ft. per sec. per sq. mile for 40 hours, and in the case of the December freshet this rate was equalled or exceeded for only 19½ hours. During this time, however, the range above the 100 was three times that of the previous month. It appears, therefore, that, in caring for freshet flows in these localities, provision must be made for a rate of run-off per square mile of at least 300 cu. ft. per sec. per sq. mile continuing for 20 hours, and that total discharges of 32 000 000 cu. ft. per sq. mile for water-sheds having an area of 400 sq. miles, and 26 000 000 cu. ft. per sq. mile for those of twice this area, delivered in from 4 to 5 days, are not uncommon. In the Northern States, if provision is made for a run-off of 6 in. per 24 hours, it is thought to be conservative. The freshet of December, 1909, delivered at about twice this rate. The curves, of course, are only a different way of presenting the data already shown in Plate XXIX and Fig. 39 (Chagres River freshet).

The Chagres River freshets are usually not a single rise and fall of the river, but a series of rises following each other more or less closely, depending on the rainfall and the portion of the drainage area in which the storm begins or is centered. The smaller rises which appear are often due principally to the fact that the larger rises have preceded so closely; and in many cases the former would not have been so noticeable except for the fact that the ground had just been soaked so thoroughly that most of the rainfall found its way very quickly into the stream. Table 37 shows the total run-off at Alhajuela, Bohio, and Gatun for the freshet of December 26th, 1909, and another of February 12th, 1910. This latter freshet is principally remarkable for occurring during the period when rainfall and stream flow are usually at the minimum condition; the former freshet occurred during the period of maximum rainfall and river flow. The columns under Item (1) show the actual discharge conditions, and those under Item (2) give the estimated discharge due to the freshet in addition to the normal river flow, estimating the latter from the gauge height just before and after the rise. In the absence of rain gauges on the area above Alhajuela, the rainfall seems to be best estimated by taking the average of the Porto Bello and Alhajuela records. In the freshet of December 26th, this rainfall was about 19.50 in., and the discharge due to the particular conditions at this time was about

PLATE XXX.
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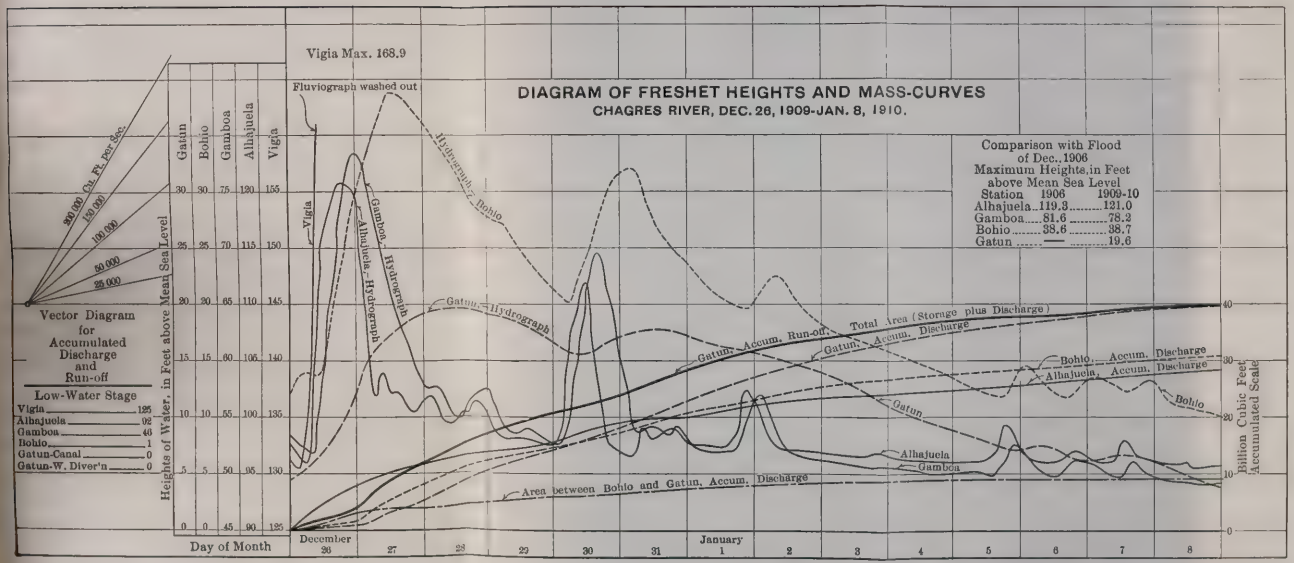
HYDROLOGY OF THE
PANAMA CANAL.



17.18 in., showing that nearly this entire rainfall was discharged during the period of 4 or 5 days succeeding. An inspection of Table 36 will show that the rise of December 26th-27th was due to rainfall which was quite general over the basin, and that of the 20th was due almost entirely to rainfall in the extreme eastern part of the territory, in the neighborhood of Porto Bello. This latter condition was also true in the freshet of February, 1910, when 4.38 in. of rain fell at Porto Bello, 1.62 in. at Alhajuela, and less than $\frac{1}{4}$ in. elsewhere in the whole basin.

TABLE 37.—Chagres River Drainage Basin.
Freshets Beginning December 26th, 1909, and February 12th, 1911.
Accumulated Discharges.

Date.	Alhajuela. Drainage area = 427 sq. miles.			Bohio. Drainage area = 779 sq. miles.			Gatun. Drainage area = 1 320 sq. miles.		
	Thousand million cubic feet.	Million cubic feet per square mile.	Total inches of run-off.	Thousand million cubic feet.	Million cubic feet per square mile.	Total inches of run-off.	Thousand million cubic feet.	Million cubic feet per square mile.	Total inches of run-off.
Item (1) Including the River Flow at Time Freshet Began.									
Dec. 26th–Jan. 8th 14 days.	28.40	66.5	28.56	31.0	39.8	17.22	40.0	30.3	13.02
Feb. 12th–Feb. 15th 3.5 days.	2.35	5.5	2.35	....	....	....	3.02	2.28	0.98
Item (2) Not Including River Flow—Actual Run-off Due to Freshet.									
Dec. 26th–Jan. 8th 14 days.	17.27	40.45	17.18	18.66	23.96	10.30	25.06	19.00	8.18
Feb. 12th–Feb. 15th 3.5 days.	1.92	4.51	1.94	....	....	....	2.41	1.83	0.78

The freshet of December 26th gave the greatest momentary discharge at all gauging stations, and that of November gave the greatest quantity of water. The comparative relations of the former are given in Table 38.

TABLE 38.

	Maximum Discharge for Short Period.		Ratio to Mean Discharge.		
	Cubic feet per second.	Cubic feet per second per square mile.	December.	Annual.	Minimum month.
Alhajuela	170 000	398	37.2	66.0	222
Gatun	124 000	94	10.0	16.4	76

Fig. 41 shows the hydrograph and discharge curves for the "dry season" freshet of February, 1910, and it is interesting to note that the total run-off at Gatun was about one-fourth of the discharge of the "wet season" freshet of December, 1909. Figs. 42 and 43 show the surface slope of the Chagres River during freshet periods, with the crest at various places. The time the crest passed the gauging stations

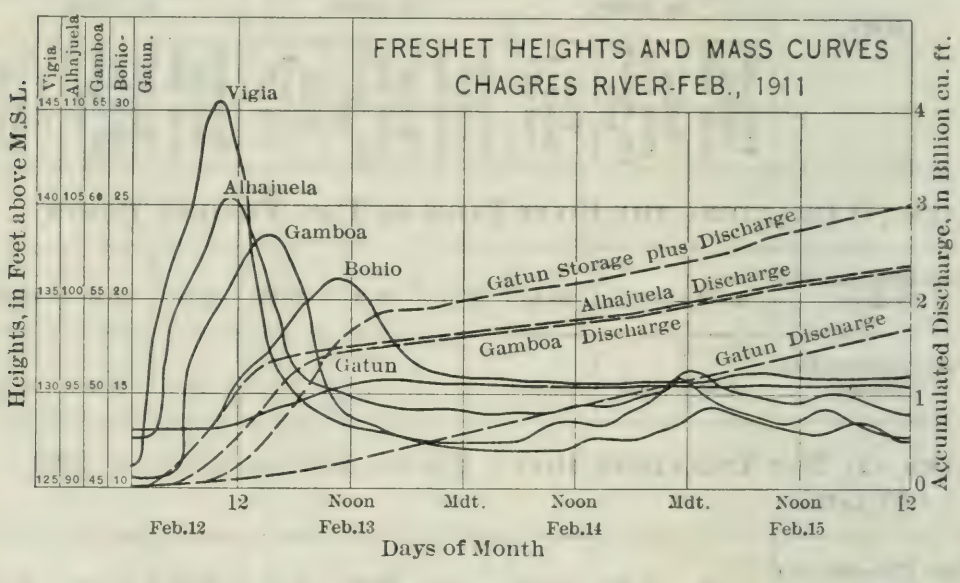


Fig. 41.

is given on Fig. 42, and from this it is possible to note the progress of the crest down the river. Table 39, for the freshet of December 26th, 1909, shows these data in concrete form.

The Gamboa station had not been established in December, 1909, but the flow past the station at Alhajuela had a mean maximum velocity of 13.57 ft. per sec., or more than four times that with which the crest moved down stream. In the lesser rise of February, 1911, the

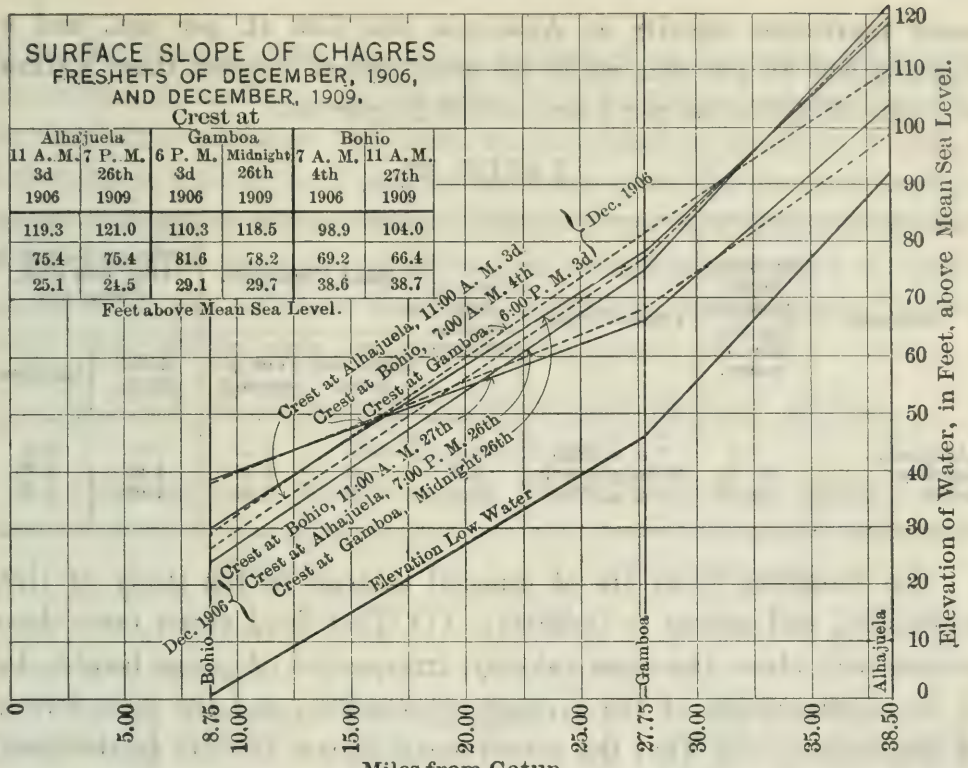

SURFACE SLOPE OF CHAGRES
FRESHETS OF DECEMBER, 1906,
AND DECEMBER, 1909.
Crest at
Alhajuela
11 A. M. 3d 1906
7 P. M. 26th 1909
Gamboa
6 P. M. 3d 1906
Midnight 26th 1909
Bohio
7 A. M. 4th 1906
11 A.M. 27th 1909
119.3 121.0 110.3 118.5 98.9 104.0
75.4 75.4 81.6 78.2 69.2 66.4
25.1 24.5 29.1 29.7 38.6 38.7
Feet above Mean Sea Level.
Crest at Alhajuela, 11:00 A. M. 3d
Crest at Bohio, 7:00 A. M. 4th
Crest at Gamboa, 6:00 P. M. 3d
Dec. 1906
Crest at Bohio, 11:00 A. M. 27th
Crest at Alhajuela, 7:00 P. M. 26th
Crest at Gamboa, Midnight 26th
Elevation Low Water
Dec. 1906
Bohio
Gamboa
Alhajuela
Elevation of Water, in Feet, above Mean Sea Level.
0 5.00 8.75 10.00 15.00 20.00 25.00 27.75 30.00 35.00 38.50
Miles from Gatun.

FIG. 42.

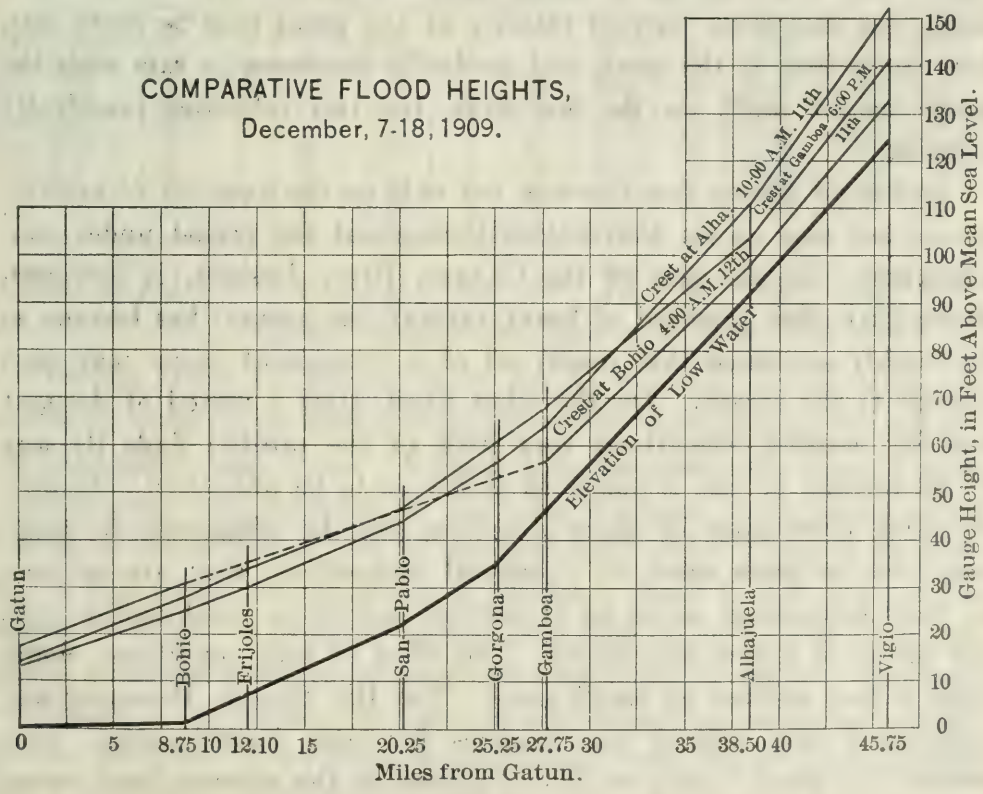

COMPARATIVE FLOOD HEIGHTS,
December, 7-18, 1909.
Crest at Alha., 10:00 A.M. 11th
Crest at Gamboa, 6:00 P.M. 11th
Crest at Bohio 4:00 A.M. 12th
Elevation of Low Water
Gatun
Bohio
Frijoles
San Pablo
Gorgona
Gamboa
Alhajuela
Vigio
Gauge Height, in Feet Above Mean Sea Level.
0 5 8.75 10 12.10 15 20.25 25.25 27.75 30 35 38.50 40 45.75
Miles from Gatun.

FIG. 43.

mean maximum velocity at Alhajuela was 7.85 ft. per sec., and at Gamboa 6.66 ft. per sec., while the crest moved between these stations at a rate of 2.35 miles per hour, or 3.45 ft. per sec.

TABLE 39.

Station.	Distance from preceding station, in miles.	Time of crest.	Time to next station, in hours.	Mean Velocity.		Surface Slope, in feet per mile. Crest at:	
				Miles per hour.	Feet per second.	Alhajuela.	Gamboa
Alhajuela.....	0.0	7 P. M. 26th	0.0	0.0	0.0	0.0	0.0
Gamboa.....	10.75	Midnight 26th	5.0	2.14	3.14	4.25	3.75
Bohio.....	19.00	11 A. M. 27th	11.0	1.73	2.54	2.68	2.56

The foregoing facts are of especial interest in the study of river hydraulics, and appear to indicate: (1) That flood crests move down stream with about the same velocity, irrespective of gauge height, due to the configuration of the average cross-section and the meanderings of the stream. (2) That the actual mean stream velocity is absolutely dependent on the gauge height. In the high rises in the Chagres River, the maximum current velocity at any point may be more than four times that of the crest, and gradually decreases in rate with the gauge height until, on the low flows, the two velocities practically coincide.

Run-off or stream flow depends, not only on the quantity of precipitation, but also on its distribution throughout the period under consideration. In the case of the Chagres River freshets, it has been shown that after a period of heavy rainfall the ground has become so thoroughly saturated that nearly all of a subsequent storm may pass quickly to the stream. On the other hand, after a period of drought and dry weather, oftentimes very little of the rainfall finds its way to the stream, or else it comes so slowly as to be unnoticed. Though excessive downpours of short duration may be disastrous to small reservoirs in some cases, as a general proposition, they are so local in their happenings as to be of interest mainly in combination with the design of sewers and drains. The effect of such conditions, therefore, is best studied on small areas. The Rio Grande Reservoir and water-shed have offered opportunities for such investigation. This reservoir is about 1 mile south of Culebra, at the extreme head-waters

of the Rio Grande. The top of the spillway is at Elevation 235.2, and the surface of the lake can be raised 3 ft. by the use of flash-boards. At the latter elevation the area of the reservoir surface is about 0.09 sq. mile, and the total area of the basin above the dam and including the reservoir is 3.15 sq. miles. The land area is wholly uninhabited, is covered with tropical vegetation and second growth, and is representative of the ordinary woodland conditions which obtain in the greater part of the river basins on the Isthmus.

TABLE 40.—Excessive Rainfall and Run-off in the Rio Grande Basin.

Drainage Area above Gauging Station = 2.36 Sq. Miles.

Rainfall.							Mean Discharge.				
(1)	(2)	(3)		(4)	(5)		(6)	(7)	(8)	(9)	(10)
Date, 1911.	Quantity, in inches.	Duration of storm.		Maximum rate, in inches.	Period of maximum rate.		Cubic feet per second.	Period, in hours.	Cubic feet per second per square mile.	Inches on water-shed.	Ratio of run-off to rainfall. Percentage.
		Hrs.	Min.		Hrs.	Min.					
April 25th.	3.12	3	15	2.70	..	..	18.75	12	7.94	0.15	5.0
May 9th...	2.08	1	55	2.06	..	..	23.03	15	9.76	0.23	11.0
May 11th..	2.97	3	15	2.45	1	00	35.40	24	15.00	0.56	19.0
May 13th..	1.50	2	35	0.61	2	10	70.80	24	30.00	1.12	72.0
May 25th..	2.73	2	4	4.04	1	00	59.20	24	25.10	0.93	34.0
May 30th..	1.25	4	30	2.85	0	38	11.77	48	4.99	0.37	29.7
July 6th...	1.45	0	50	2.60	0	32	20.82	48	8.82	0.66	45.2
July 8th...	0.98	1	10	1.61	0	32	17.76	48	7.52	0.56	57.0
Aug. 22d..	2.13	4	20	15.53	0	8	18.77	72	7.95	0.89	41.7
July 21st..	0.77	..	..	....	..	..	0.50	5	0.24	0.0039	0.21
July 23d..	0.82	1	15	1.56	0	25	2.07	12	0.88	0.016	2.00

Though there are several small brooks running directly into the reservoir, most of the area is tributary to the main stream for quite a distance above high water. A short distance above this point a river stage register was established, the area tributary from above being 2.36 sq. miles, or about 78% of the total land area. There is an automatic rain gauge on the shore of the lake, about $\frac{3}{4}$ mile east of the register station. A gauging section was established near the register, and a sufficient number of gaugings were made to fix the discharge curve and rating table. In Table 40 are shown the record and distribution for some of the most severe storms which have occurred since the station was established. Attention is called particularly to the

progressive increase in the ratio of run-off to rainfall, Column 10. Near the end of the dry season there had been only 3.34 in. of rain since February 15th, and 2.38 in. of this had been in a sudden downpour. The ground, therefore, was parched and dry, and the vegetation eager for moisture. On April 25th, 3.12 in. of rain fell in 3 hours 15 min., but only 5% appeared as run-off in the stream. As the rains became more frequent and the depleted ground storage became filled, a progressively greater quantity appeared in the stream flow until about July. A ratio of about 45% had been reached then, and the percentage of run-off afterward oscillated above and below this mark, depending on the rainfall and the quantity of water stored in the ground. The ratio of 72% appearing from the storm of May 13th is due to the heavy rainfalls just previous, which had not fully been cared for either by storage or surface flow. The data concerning the storms of July 21st and 23d are also of interest as an indication of the almost complete loss of a considerable quantity of water to stream flow and water use. This condition is due to the excessively dry period of June and July, when the rainfall was considerably below normal. It is indirect proof of the large water requirements of vegetation, and may be considered as a total loss to the lake. A study of the ground-water conditions showed that little, if any, of this rain appeared as run-off, although the run-off probably benefited indirectly by a lessened demand on the ground reservoir for vegetation.

The storm of May 25th was rather notable for the quantity of rain falling in a short time, and the conditions of run-off are those which ordinarily obtain from storms of this character. It occurred at a time when the ground reservoir was considerably filled and the demands of vegetation would not be out of the ordinary. Certain facts concerning this storm are shown by Fig. 44. The total rainfall was 2.73 in., of which a little more than 2 in. fell in 1 hour. The rain began at 12.12 P. M., and 0.29 in. fell in 26 min., no run-off being appreciable on the river gauge. The rain again began at 2.54 P. M., and at 4.32 P. M. 2.44 in. had fallen. At 3 P. M. the river gauge registered a flow of 1.3 cu. ft. per sec., and at this time the discharge began to increase. The maximum flow came at 4.15 P. M., the gauge showing a rise of 5.31 ft. and a discharge at the rate of 380 cu. ft. per sec. The stream flow did not return to its original discharge until 3 P. M. of May 30th, a period of 5 days. From 4.30 P. M. of the 25th to 3 P. M. of May 30th

0.18 in. of rain fell between 2 A. M. and 10 P. M. of the 26th. As the 0.15 in. of rain that fell on May 25th, prior to the principal downpour, did not affect the stream flow, it is probable that the 0.18 in.

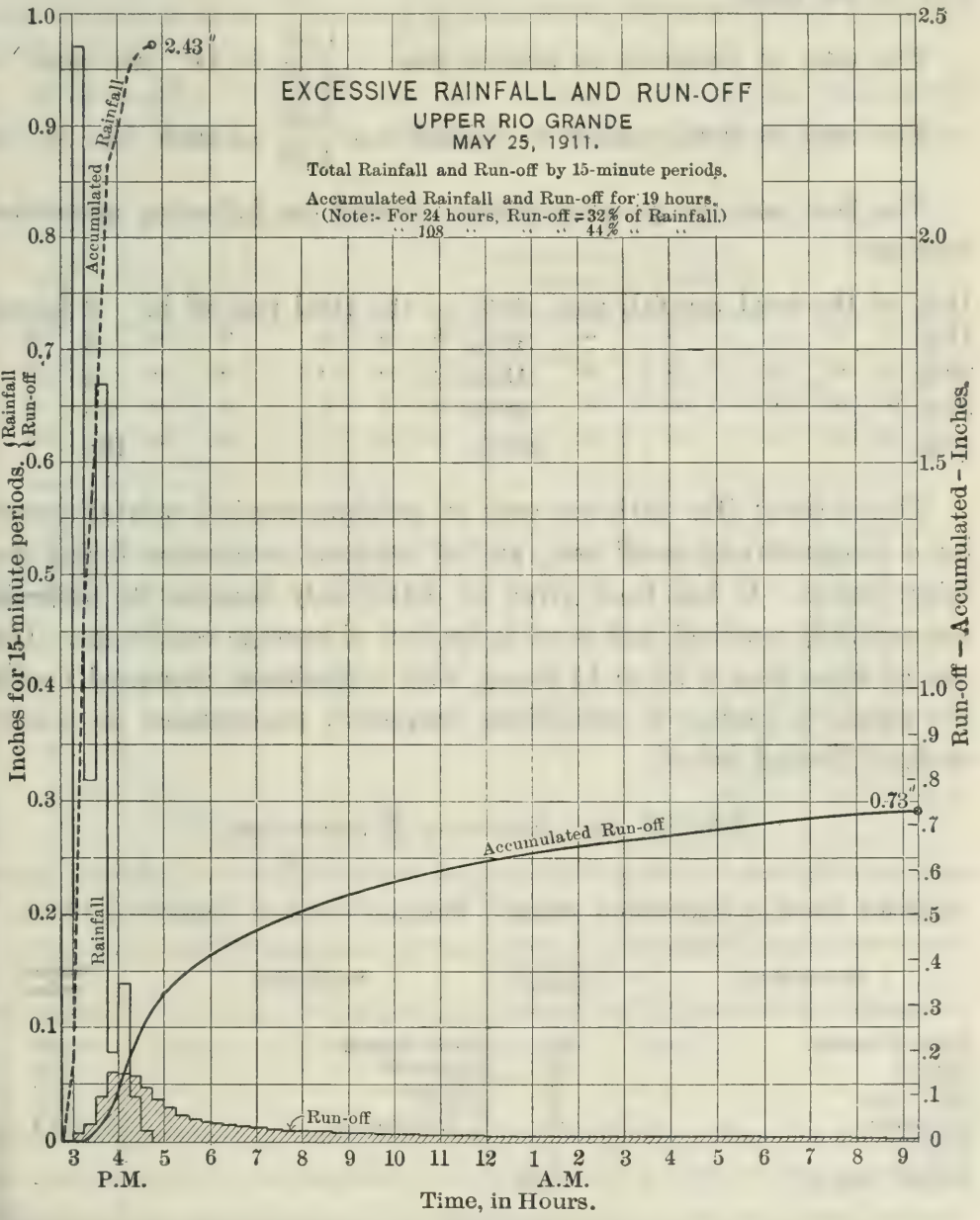


FIG. 44.

falling 9½ hours later also had so little effect as to be negligible. Assuming this condition, the rainfall between 2.54 P. M. and 4.30 P. M., May 25th, caused a run-off, which, beginning at 3 P. M., May 25th, reached a maximum at 4.15 P. M. of the same day, and its effect was

noted in the stream flow up to 3 A. M. of May 30th. That is, 2.43 in. of rain falling in 1 hour 38 min., at an average rate of 1.76 in. per hour, caused a run-off of 1.07 in. in 108 hours, at an average rate of about 0.01 in. per hour.

The ratio of discharge to rainfall was.... $\frac{1.07}{2.43} = 44$ per cent.

The ratio of time, run-off to rainfall was $\frac{1.08}{1.63} = 66.4$ " "

The flow past the gauging station showed the following quantities passing:

12%	of the	total	rainfall	and		30%	of the	total	run-off	in	2	hours.
18%	"	"	"	"	"	40%	"	"	"	"	3	"
20%	"	"	"	"	"	45%	"	"	"	"	4	"
27%	"	"	"	"	"	60%	"	"	"	"	10	"
44%	"	"	"	"	"	100%	"	"	"	"	108	"

The storm of May 25th was only an ordinary tropical rainfall covering a comparatively small area, and of common occurrence during the rainy season. It has been given in detail only because its progress was carefully watched, and is an indication of average conditions. The rise of more than 5 ft. in 1¼ hours, with a discharge increased nearly 300 times, is similar to conditions frequently encountered on water-sheds of limited extent.

TABLE 41.—AREAS OF WATER-SHEDS.

DRAINAGE NORTH OF CONTINENTAL DIVIDE.		DRAINAGE SOUTH OF CONTINENTAL DIVIDE.	
Water-shed.	Square miles.	Water-shed.	Square miles.
Upper Chagres	212	Pedro Miguel	10.9
Pequini	189	Cardenas	10.6
Chilibre	54	Caimeto	18.6
Gatuncillo	38	Cocoli	12.4
La Puente	22	L. Rio Grande	10.4
Frijoles	7		
Frijolito	10.2		
Frijoles Grande	2.75		

The relation between the rainy- and dry-season discharges of streams is particularly interesting in its bearing on water supply, and the use to which it can be put in regard to stream flow and reservoir storage. Figs. 45, 46, and 47 show these conditions graphically, and

little description seems necessary. It appears that the extreme low flows last for comparatively short periods, and though in many cases the discharge of the stream through the dry months is very well sustained from ground storage, in other cases rivers draining considerable areas almost cease to flow. The areas of the several drainage basins are given in Table 41.

It is worthy of notice that the streams south of the continental divide and flowing into the Pacific Ocean generally have a much less

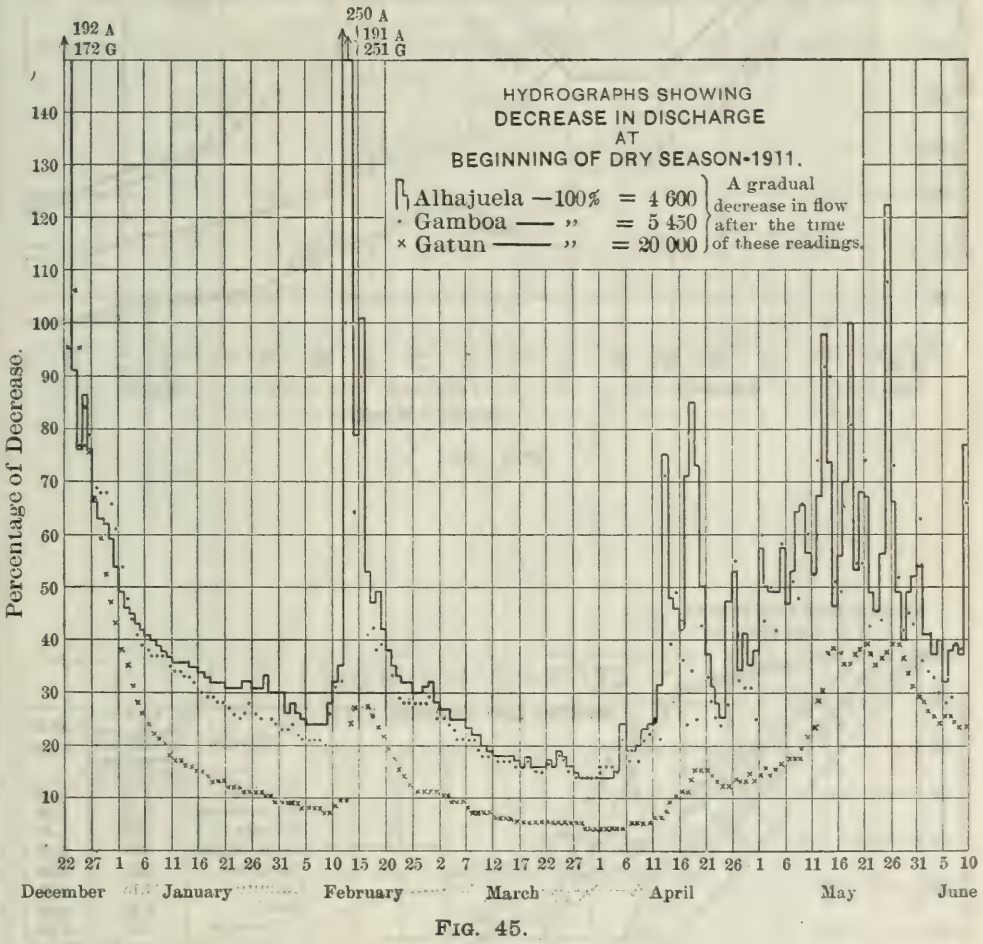


FIG. 45.

dry-season discharge than those on the northerly side. As mentioned in the discussion on rainfall, this condition is due to the prevailing winds and to the fact that the northerly side is the windward side of the cordillera.

The relations of rainfall and run-off in the basin seem to be best presented in diagrammatic form. The run-off, as measured and estimated at the several principal stations, has been given in Tables 19

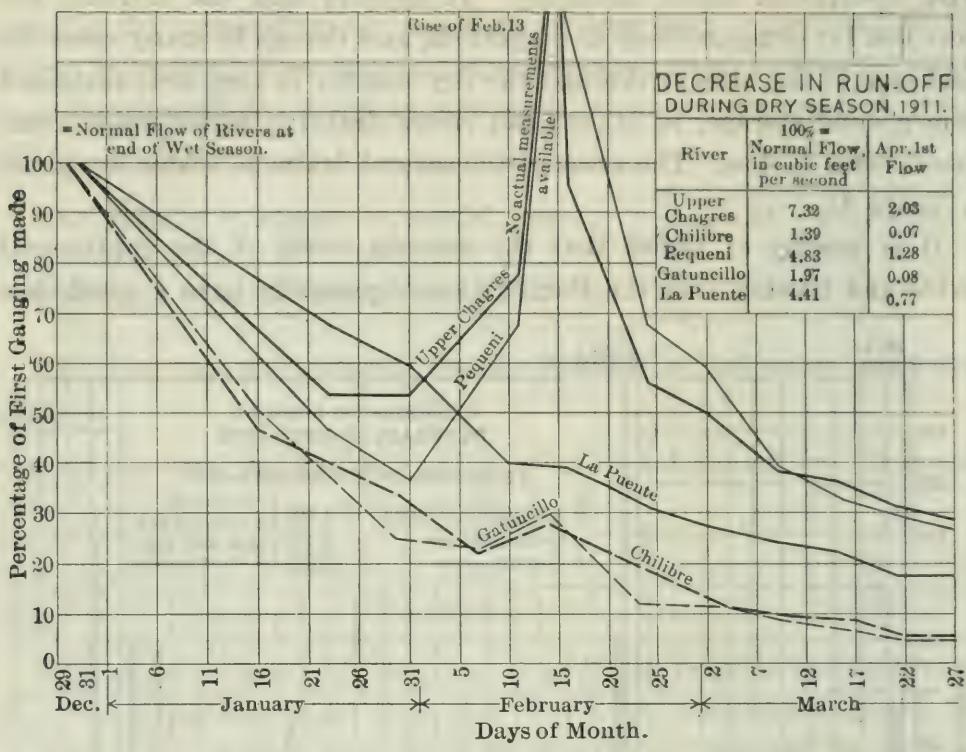

Rise of Feb. 13
No actual measurements available
DECREASE IN RUN-OFF
DURING DRY SEASON, 1911.
River
100% = Normal Flow, in cubic feet per second
Apr. 1st Flow
Upper Chagres 7.32 2.03
Chilibre 1.39 0.07
Pequeni 4.83 1.28
Gatuncillo 1.97 0.08
La Puente 4.41 0.77
= Normal Flow of Rivers at end of Wet Season.
Upper Chagres
Pequeni
La Puente
Gatuncillo
Chilibre
Percentage of First Gauging made
Dec.
January
February
March
Days of Month.

FIG. 46.

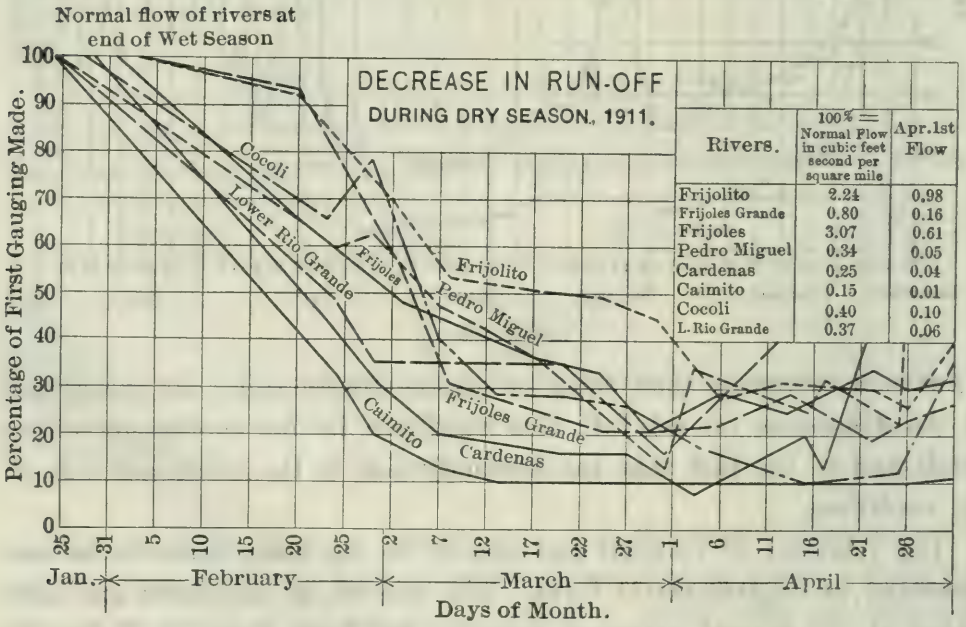

Normal flow of rivers at end of Wet Season
DECREASE IN RUN-OFF
DURING DRY SEASON, 1911.
Rivers.
100% = Normal Flow in cubic feet second per square mile
Apr. 1st Flow
Frijolito 2.24 0.98
Frijoles Grande 0.80 0.16
Frijoles 3.07 0.61
Pedro Miguel 0.34 0.05
Cardenas 0.25 0.04
Caimito 0.15 0.01
Cocoli 0.40 0.10
L. Rio Grande 0.37 0.06
Cocoli
Lower Rio Grande
Frijoles
Frijolito
Pedro Miguel
Frijoles Grande
Caimito
Cardenas
Percentage of First Gauging Made.
Jan.
February
March
April
Days of Month.

FIG. 47.

to 32, inclusive, and the rainfall or record in Tables 7 to 11. Figs. 48 to 53, inclusive, show the run-off in the Chagres Basin for the average year and for other critical periods at Alhajuela and Gatun. Before studying the river relations, it was necessary to have the quantity of rainfall on the area considered. An estimate of this, based on the actual records from the stations, weighted for their relative influence, have been prepared. The values are given in Tables 44 and 45.

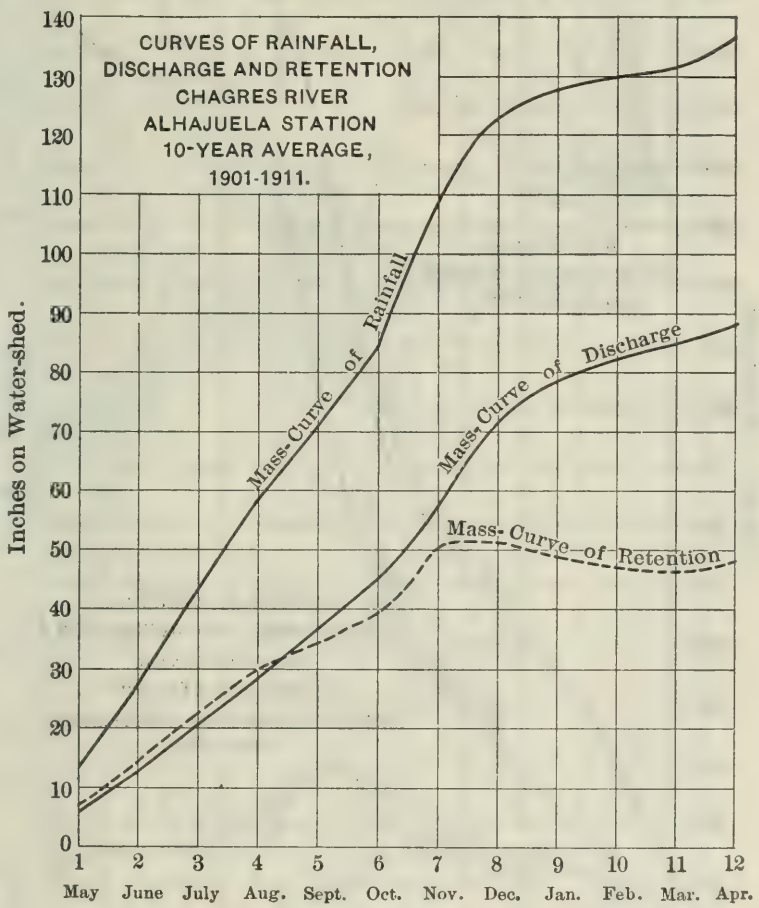


FIG. 48.

In many cases the rainfall records were lacking, and these were estimated from present annual ratios between the nearest station with a long-time record and the station where a value was required.

Attention is called to the fact that, although the run-off is a function of the rainfall, and would in time absolutely cease if the latter should stop, the quantity of the former does not vary directly with the quantity of the latter. During comparatively short periods of

little or no rainfall the run-off is many times in excess of the precipitation, and for longer periods the run-off decreases with the rainfall. This decrease, however, is not directly proportional, but is in some more complex ratio, depending on the character of the water-shed and

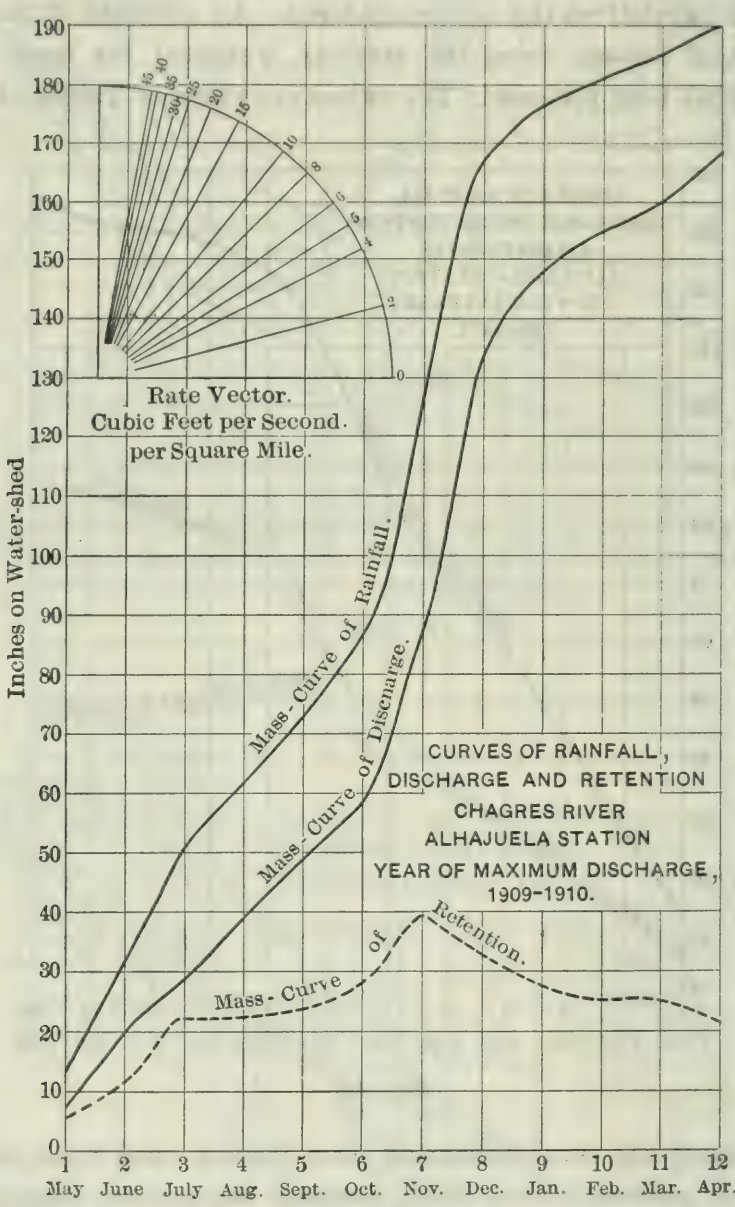
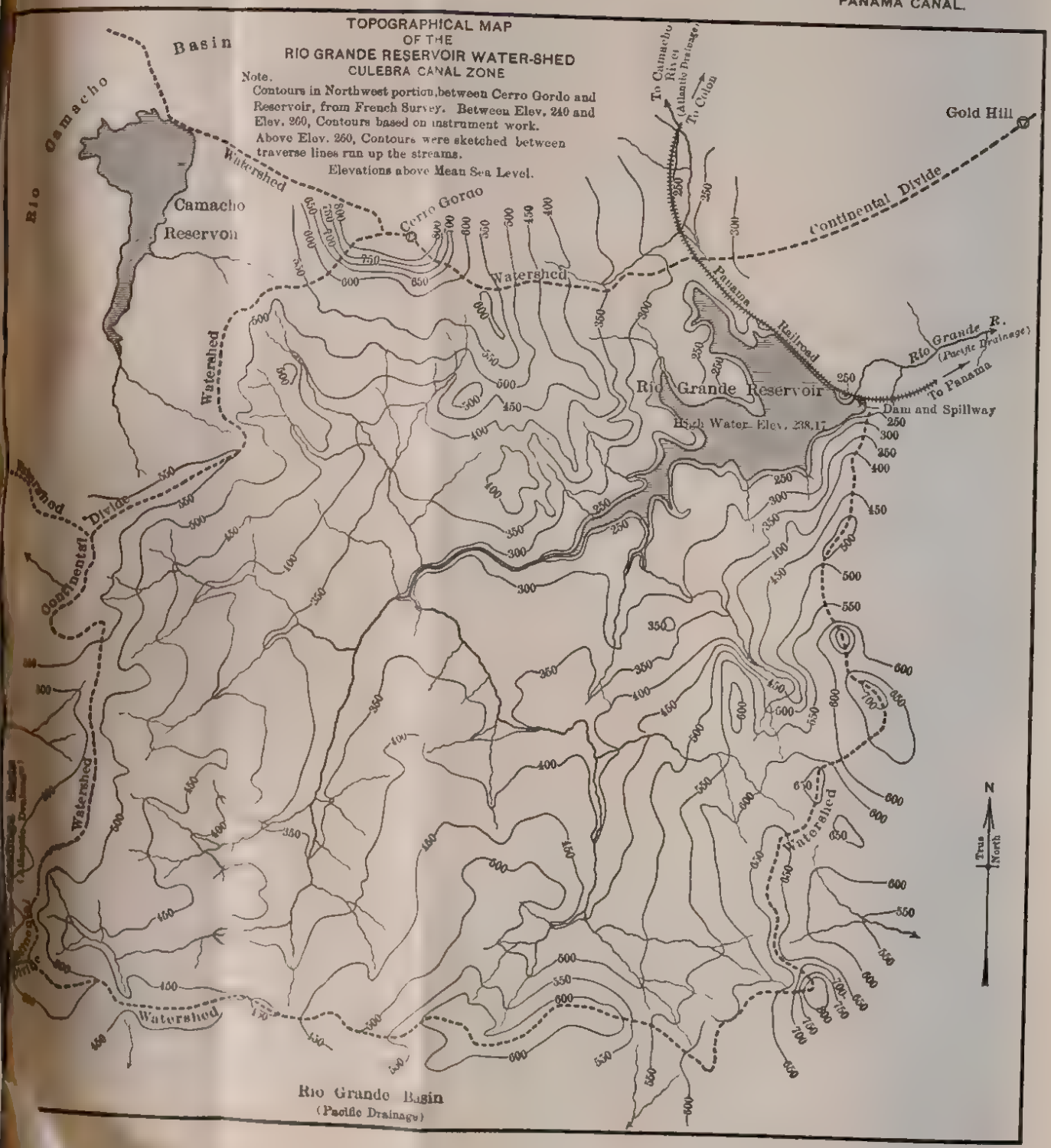

Inches on Water-shed
Rate Vector.
Cubic Feet per Second.
per Square Mile.
Mass-Curve of Rainfall.
Mass-Curve of Discharge.
Mass-Curve of Retention.
CURVES OF RAINFALL, DISCHARGE AND RETENTION
CHAGRES RIVER
ALHAJUELA STATION
YEAR OF MAXIMUM DISCHARGE, 1909-1910.
May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr.

FIG. 49.

its vegetation. For example, a rainfall of 25% less than normal will produce a certain run-off during the year, under certain conditions, but a rainfall of 50% of the normal for this period would probably yield much less than 50% of the normal run-off. For this reason, it


TOPOGRAPHICAL MAP
OF THE
RIO GRANDE RESERVOIR WATER-SHED
CULEBRA CANAL ZONE
Note.
Contours in Northwest portion, between Cerro Gordo and Reservoir, from French Survey. Between Elev. 240 and Elev. 260, Contours based on instrument work.
Above Elev. 260, Contours were sketched between traverse lines run up the streams.
Elevations above Mean Sea Level.
Rio Camacho Basin
Camacho Reservoir
Watershed
Cerro Gordo
To Camacho Rivers (Atlantic Drainage)
To Colon
Gold Hill
Continental Divide
Panama Railroad
Rio Grande R. (Pacific Drainage)
To Panama
Rio Grande Reservoir
Dam and Spillway
Rio Grande Basin
(Pacific Drainage)
N
True North

is very unsafe to base calculations of yield on short-time observations, and apply the relation thus established to minimum conditions of rainfall existing at some former period. For the purpose of illustrating these conditions, Tables 42 and 43 and Fig. 53 are given for the whole area of the basin above Gatun (1 320 sq. miles), and for the area above Alhajuela (427 sq. miles).

The year of lowest rainfall in this decade is 1902-03, with 93.38 in. The river discharge for this year was 48.87, or 52.4 per cent.

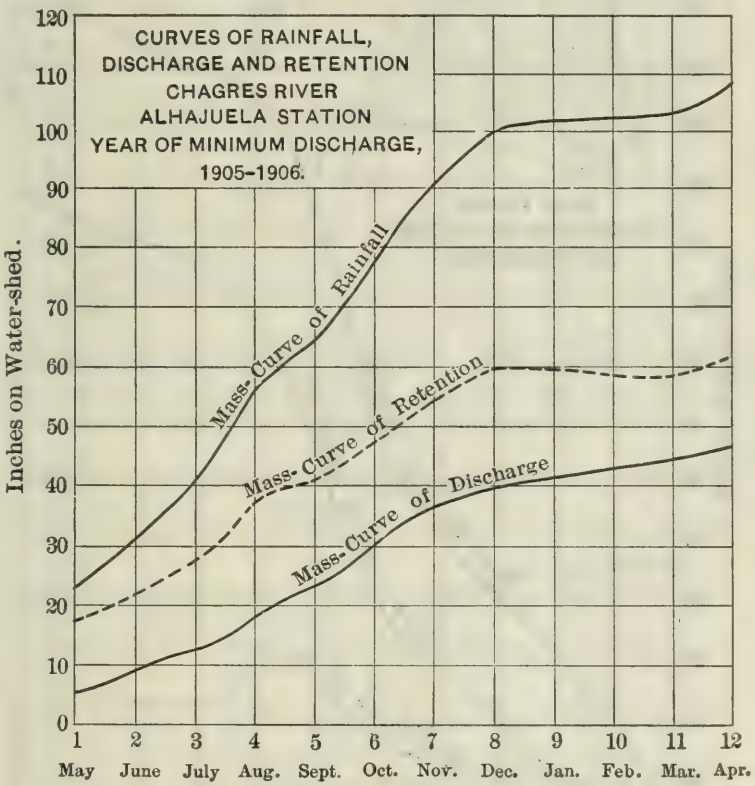


FIG. 50.

April, 1905, was quite dry, and the heavy rainfall of 20.22 in. in May produced only a moderate run-off, although it increased considerably the total rainfall for the year.

A comparison of the retention curves and discharge tables given in this paper indicates in some degree the quantity of water required by vegetation. In the year of plenteous rainfall, besides actual precipitation, the air contains more water vapor, which, being absorbed by the leaves and grasses, gives them a supply of moisture in lieu of actual rainfall. In the years of less rainfall these demands must be met from

ground storage, and thus a large quantity of actual precipitation must be taken, to the consequent detriment of stream flow.

In the year of heavy rainfall the subterranean reservoir is kept full, and rainfall reaching the surface of the ground, having no opportunity to delay, is delivered quickly to the stream with minimum loss.

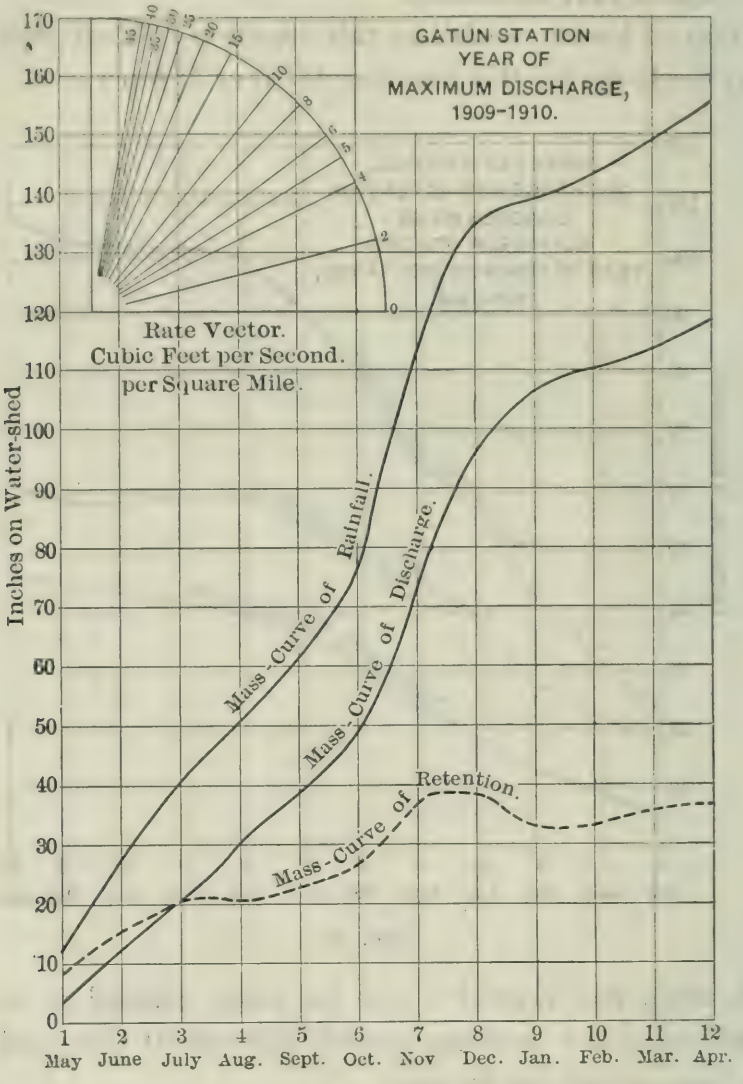


FIG. 51.

What proportion of the stream flow is due to sub-surface flow is a complex question, and each water-shed has a coefficient of its own that must be applied to any general statement. The quantity depends on the character of the soil, the underlying strata, the topography, and the surface cover of vegetation. That considerable quantities of

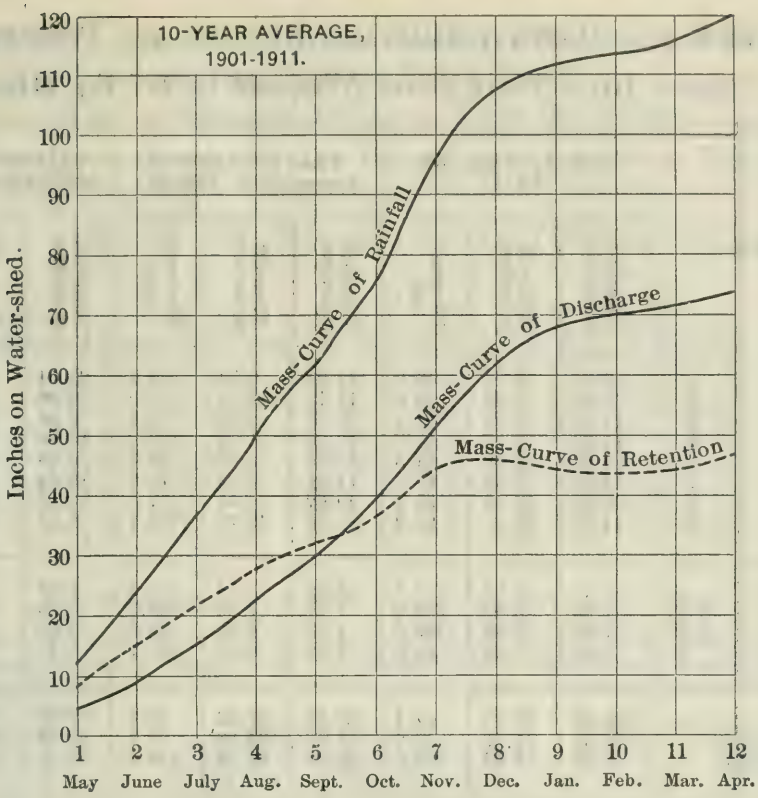

10-YEAR AVERAGE,
1901-1911.
Inches on Water-shed.
Mass-Curve of Rainfall
Mass-Curve of Discharge
Mass-Curve of Retention
0
10
20
30
40
50
60
70
80
90
100
110
120
1 2 3 4 5 6 7 8 9 10 11 12
May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr.

FIG. 52.

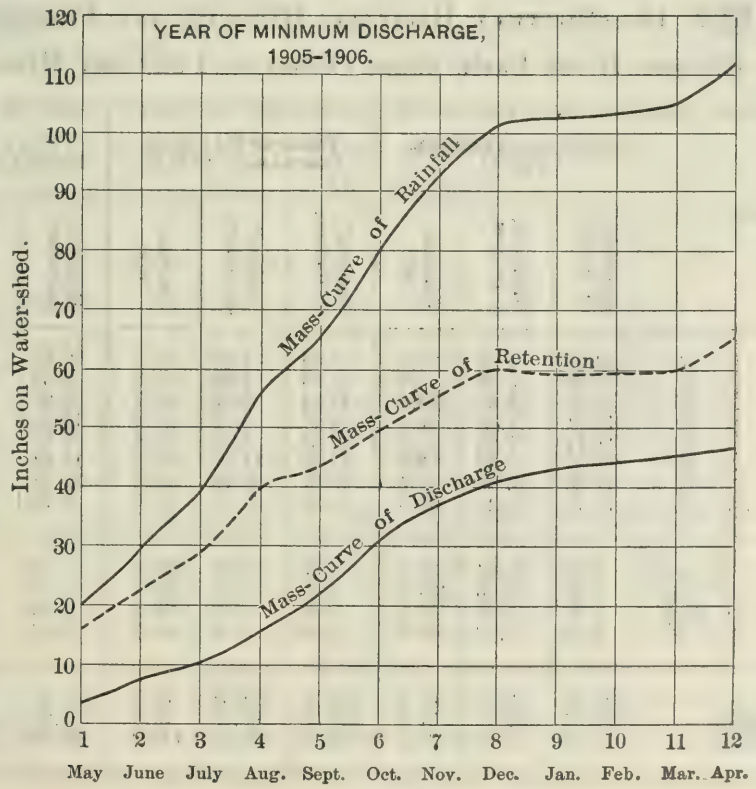

YEAR OF MINIMUM DISCHARGE,
1905-1906.
Inches on Water-shed.
Mass-Curve of Rainfall
Mass-Curve of Retention
Mass-Curve of Discharge
0
10
20
30
40
50
60
70
80
90
100
110
120
1 2 3 4 5 6 7 8 9 10 11 12
May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr.

FIG. 53.

TABLE 42.—Monthly Rainfall, Run-off, and Percentage.

Chagres River Basin above Alhajuela = 427 Sq. Miles.

Month.		10-Year Average, 1901-11.			Year of Maximum Discharge, 1909-10.			Year of Minimum Discharge, 1905-06.		
		Rainfall, in inches.	Run-off, in inches.	Percentage.	Rainfall, in inches.	Run-off, in inches.	Percentage.	Rainfall, in inches.	Run-off, in inches.	Percentage.
May	Rainy Season.	13.08	6.01	46.0	12.00	6.54	54.5	25.04	5.21	22.6
June		13.94	6.49	46.6	18.72	12.73	68.0	8.36	4.16	49.8
July		16.03	7.76	51.6	19.98	8.91	44.6	9.08	3.37	37.2
Aug		15.36	7.95	51.8	10.99	11.04	100.2	15.78	5.96	37.8
Sept		12.56	8.08	64.3	10.74	9.31	86.7	6.00	4.50	56.3
Oct		13.86	8.58	61.7	14.01	9.89	70.6	13.64	7.25	53.2
Nov		23.33	12.49	53.6	40.62	29.55	72.8	13.00	5.92	45.5
Dec		14.27	13.91	96.9	40.54	46.71	105.1	8.56	3.41	39.8
Jan	Dry Season.	5.18	7.07	136.5	8.52	13.64	160.1	1.55	1.85	133.4
Feb		1.85	3.81	205.9	4.24	7.00	165.2	0.85	1.50	176.5
Mar		1.68	2.30	137.0	4.53	4.39	96.9	0.60	1.18	196.9
Apr		5.22	3.34	64.0	5.06	8.21	162.2	5.10	1.86	36.5
Total		136.36	87.79	64.4	189.95	167.92	88.4	107.56	46.17	42.89
8 wet months		122.43	71.27	58.2	167.60	134.68	80.4	99.46	39.78	40.00
4 dry months		13.93	16.52	118.6	22.35	33.24	148.6	8.10	6.39	78.90

TABLE 43.—Monthly Rainfall, Run-off, and Percentage.

Chagres River Basin above Gatun = 1 320 Sq. Miles.

Month.		10-Year Average, 1901-11.			Year of Maximum Discharge, 1909-10.			Year of Minimum Discharge, 1905-06.		
		Rainfall, in inches.	Run-off, in inches.	Percentage.	Rainfall, in inches.	Run-off, in inches.	Percentage.	Rainfall, in inches.	Run-off, in inches.	Percentage.
May	Rainy Season.	12.49	4.02	33.3	12.44	3.68	29.6	20.22	3.99	19.7
June		11.82	5.23	44.2	15.24	8.35	54.8	9.60	3.63	37.8
July		12.81	6.30	49.2	13.44	8.69	64.7	9.25	2.70	29.2
Aug		13.51	7.15	52.9	9.81	9.59	97.8	16.96	5.49	32.4
Sept		11.74	7.63	64.9	10.66	8.38	78.6	8.90	5.41	60.8
Oct		13.69	8.94	65.3	14.96	11.00	73.5	15.64	9.82	62.8
Nov		20.33	12.50	61.5	35.01	24.03	69.8	12.33	6.47	52.5
Dec		11.18	10.40	93.0	23.08	22.54	97.7	8.75	4.04	46.2
Jan	Dry Season.	4.14	5.54	133.8	4.35	10.26	235.9	1.25	1.89	151.2
Feb		1.69	2.26	133.8	4.64	4.01	86.4	1.06	1.10	103.8
Mar		1.96	1.46	74.5	5.61	2.76	49.2	1.15	0.87	75.7
Apr		4.70	2.13	45.3	5.96	5.14	86.2	6.83	1.37	20.1
Total		120.06	73.56	61.2	155.20	118.48	76.3	111.94	46.78	39.2
8 wet months		107.57	62.17	57.9	134.64	96.31	71.3	101.65	41.55	40.9
4 dry months		12.49	11.39	91.1	20.56	22.17	110.2	10.29	5.23	50.8

TABLE 44.—Average Estimated Monthly Rainfall, in Inches, on the Chagres River Basin.

Area of Chagres Basin = 1 320 Sq. Miles.

The yearly totals are for the run-off year beginning May 1st.

Month.	1901–1902.	1902–1903.	1903–1904.	1904–1905.	1905–1906.	1906–1907.	1907–1908.	1908–1909.	1909–1910.	1910–1911.	Monthly average.
May	10.21	11.93	12.55	10.09	22.27	10.64	7.59	19.03	11.99	11.51	12.78
June	11.06	7.89	10.72	15.16	8.90	12.37	14.90	12.24	15.12	13.03	12.14
July	10.72	14.79	16.56	12.76	9.44	16.90	12.45	9.50	14.49	18.21	13.58
August	15.50	8.88	14.31	9.73	17.84	18.61	14.72	16.88	9.54	14.06	14.01
September	13.91	11.75	11.30	14.03	8.88	12.97	11.17	10.31	10.76	16.14	12.12
October	16.19	14.37	12.39	8.72	16.10	12.81	17.61	12.01	15.09	14.37	13.97
November	29.20	16.09	23.24	20.89	12.81	22.82	11.85	18.80	35.74	20.69	21.21
December	6.29	6.66	17.27	5.82	9.00	15.35	4.78	8.96	26.20	19.44	11.98
January	12.90	1.07	5.81	6.49	1.32	1.92	2.23	8.01	5.14	0.50	4.54
February	0.58	0.27	1.01	1.07	1.06	1.40	0.78	4.59	4.94	2.70	1.84
March	1.91	0.79	1.66	0.67	1.21	1.96	1.86	2.45	5.97	1.31	1.98
April	4.74	1.46	11.09	1.90	6.39	0.90	3.05	8.44	6.16	4.77	4.89
Year	133.21	95.95	137.91	107.33	115.22	128.65	102.99	131.22	161.14	136.73	125.04

The average monthly rainfall over the Chagres River Basin was computed from the following records: The records from each section are given weights corresponding to the area of the section.

Section.	Rainfall Records.	Weight.
No. 1	Porto Bello + Alhajuela ÷ 2	427
No. 2	Alhajuela + Gamboa ÷ 2	132
No. 3	Culebra + Gamboa ÷ 2	32
No. 4	Gamboa + Bohio ÷ 2	188
No. 5	Trinidad	314
No. 6	Monte Lirio	127
No. 7	Bohio + Gatun ÷ 2	100
Total		1 320

TABLE 45.—Average Monthly Rainfall, in Inches, on the Chagres River Basin above Alhajuela.

Area = 427 Sq. Miles.

Month.	(1901)	(1902)	(1903)	(1904)	(1905)	(1906)	(1907)	(1908)	(1909)	(1910)	Monthly average.
May	11.71	12.42	12.90	10.57	23.04	10.98	6.38	20.38	12.00	10.40	13.08
June	12.74	8.80	11.93	19.00	8.36	14.28	17.14	14.02	18.72	14.44	13.94
July	10.04	16.65	20.05	15.92	9.08	21.32	17.54	9.83	19.98	19.90	16.03
August	16.57	8.25	16.56	10.52	15.78	24.22	16.86	16.81	10.99	17.07	15.36
September	15.20	11.81	11.25	15.76	8.00	12.94	13.09	11.02	10.74	15.80	12.56
October	18.86	14.32	13.88	9.18	13.64	12.26	18.86	11.18	14.01	12.46	13.86
November	30.72	16.76	25.41	23.22	13.00	26.94	11.54	26.14	40.62	18.98	23.33
December	5.74	6.59	19.96	5.83	8.56	14.94	6.46	14.87	40.54	19.19	14.27
	(1902)	(1903)	(1904)	(1905)	(1906)	(1907)	(1908)	(1909)	(1910)	(1911)	
January	12.36	0.71	5.78	6.21	1.55	1.55	2.85	11.81	8.52	0.49	5.18
February	0.54	0.30	0.40	0.74	0.85	0.86	0.75	5.26	4.24	4.60	1.85
March	1.62	0.78	1.30	1.18	0.60	1.41	2.28	1.92	4.53	1.20	1.68
April	5.55	1.24	10.47	2.12	5.10	0.76	3.08	13.05	5.06	5.74	5.22
Year	141.65	98.63	149.89	120.25	107.56	142.46	116.83	156.29	189.95	140.27	136.36

The monthly values are equal to the mean of the rainfall records at Alhajuela and Porto Bello.

The annual totals are for the run-off year beginning May 1st.

TABLE 46.—(*Continued.*)

April 6..	0.22	+ 4.01	5.65	4.23	1.42	1.64	0.170	0.52	25.1	3.9	
April 12..	0.70	+ 2.77	5.84	3.47	2.37	3.07	0.224	2.38	40.6	12.0	$\frac{0.958}{6.76} = 14.2\%$
April 18..	0.39	+ 4.10	5.83	4.49	1.34	1.73	0.126	0.05	23.0	6.7	
April 24..	0.89	+ 4.20	5.62	5.09	0.53	1.42	0.103	0.31	9.4	15.8	
April 30..	2.40	+ 0.65	5.93	3.05	2.88	5.28	0.385	3.50	48.5	40.5	
May 6..	0.50	+ 2.50	5.83	3.00	2.83	3.33	0.243	2.24	48.5	8.6	
May 12..	7.04	— 13.29	5.61	— 6.25	11.86	18.90	1.377	5.23	211.2	125.5	$\frac{6.086}{16.65} = 36.6\%$
May 18..	13.32	— 11.60	5.32	1.72	3.60	16.92	1.233	2.25	67.7	250.3	
May 24..	6.06	— 5.01	5.25	1.05	4.20	10.26	0.747	2.75	80.0	115.4	
May 30..	11.11	— 27.50	6.62	— 16.39	23.01	34.12	2.486	4.18	347.4	167.8	
June 5..	3.71	— 1.91	5.55	5.62	0.93	4.64	0.338	0.53	14.2	56.6	
June 11..	0.93	+ 2.00	5.40	2.93	2.47	3.40	0.248	1.04	45.7	17.2	
June 17..	0.54	+ 2.15	4.66	2.69	1.97	2.51	0.183	0.54	42.3	11.6	$\frac{1.212}{4.27} = 28.4\%$
June 23..	0.15	+ 3.11	5.09	3.26	1.83	1.98	0.144	0.54	36.0	2.9	
June 29..	1.67	+ 0.83	4.94	2.50	2.44	4.11	0.300	1.62	49.4	33.8	
July 5..	1.48	+ 1.27	4.99	2.75	2.24	3.72	0.271	1.26	44.9	29.7	
July 11..	10.84	— 10.51	6.19	0.33	5.86	16.70	1.217	2.49	94.7	175.2	
July 17..	1.37	+ 2.71	6.12	4.09	2.03	3.40	0.248	0.19	33.2	22.4	$\frac{2.205}{6.67} = 33.1\%$
July 23..	0.50	+ 2.60	5.48	3.10	2.38	2.88	0.210	2.06	43.4	9.1	
July 29..	0.74	+ 1.83	5.38	2.55	2.83	3.57	0.260	0.67	52.6	13.4	
Aug. 4..	5.41	— 6.81	5.90	— 1.40	7.30	12.71	0.927	2.66	123.8	91.7	
Aug. 10..	2.26	+ 1.44	6.44	3.70	2.74	5.00	0.365	1.44	42.5	35.1	
Aug. 16..	1.84	+ 2.22	5.46	4.06	1.40	3.24	0.236	0.99	25.6	33.7	$\frac{2.273}{8.16} = 27.9\%$
Aug. 22..	3.19	— 0.94	5.20	2.25	2.95	6.14	0.448	2.27	56.7	61.3	
Aug. 28..	2.61	+ 1.38	5.45	3.99	1.46	4.07	0.297	0.80	26.8	47.9	
Sept. 3..	2.89	+ 0.39	5.19	3.28	1.91	4.80	0.350	1.70	36.8	55.6	
Sept. 9..	6.79	— 4.16	5.56	2.63	2.93	9.72	0.709	1.69	52.7	122.1	

* Constant (0.0729):—c. f. s. to inches on water-shed (3.06 sq. mi.) for 6-day period.
† The mean discharge in inches for the monthly period divided by the total rainfall in inches.

TABLE 46.—Rio Grande Reservoir Data.

Elevation of Spillway = 235.2.

SUPPLY:
Main stream..... } = { Consumption (V. meter).
Brooks..... } = { Evaporation (4-ft. pan).
Ground-water..... } = { Waste (weir discharge).
Flood storage..... } = { Seepage.

WATER-SHED: Sq. Mi.
Total area..... 3.15
Water area..... 0.09
Land above gauge station..... 2.36
Land below gauge station..... 0.70

Period ending.	Supply.		Discharge.	Surface + storage, in cubic feet per second.	Ground-water, in cubic feet per second.	Run-off Surface + Ground Flow:		Rainfall, in inches.	Percentages.		
	Surface flow, in cubic feet per second.	Storage, in cubic feet per second. + Decr. — Incr.	Consumption, Evaporation + Weir, in cubic feet per second.	Columns (2) + (3).	Columns (4) — (5).	In cubic feet per second. Columns (2) + (6).	In inches. Column 7 × 0.0729.*		Ground-water to total discharge. Columns (6) — (4).	Surface supply to total discharge. Columns (2) — (4).	Run-off to rainfall. Columns (8) — (9).†
					Mean Discharge.						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Jan. 30..	2.65	+ 3.66	6.58	6.31	0.27	2.92	0.213	0.00	4.1	40.3	
Feb. 5..	1.93	+ 3.49	6.30	5.42	0.88	2.81	0.205	0.03	14.0	30.6	$\frac{0.945}{0.88} = 107.4\%$
Feb. 11..	1.69	+ 3.43	5.85	5.12	0.73	2.42	0.176	0.39	12.5	28.9	
Feb. 17..	1.62	+ 3.16	5.77	4.78	0.99	2.61	0.190	0.46	17.2	28.1	
Feb. 23..	1.24	+ 3.48	5.75	4.72	1.03	2.27	0.161	0.00	17.9	21.6	
Mar. 1..	0.93	+ 3.56	5.38	4.49	0.89	1.82	0.133	0.03	16.5	17.3	
Mar. 7..	0.91	+ 4.08	5.53	4.99	0.54	1.45	0.106	0.00	9.8	16.5	
Mar. 13..	0.91	+ 4.15	6.06	5.06	1.00	1.91	0.139	0.00	16.5	15.0	$\frac{0.650}{0.09} = 723\%$
Mar. 19..	0.89	+ 4.41	5.77	5.30	0.47	1.36	0.099	0.00	8.1	15.4	
Mar. 25..	0.61	+ 4.46	5.70	5.07	0.63	1.24	0.090	0.06	11.1	10.7	
Mar. 31..	0.32	+ 4.50	5.64	4.82	0.82	1.14	0.083	0.00	14.5	5.7	

water are received in this way is an undisputed fact. Table 46 shows some conditions that have obtained on the Rio Grande water-shed. No attempt is made to discuss the subject in this paper, the intention being to make further study before presenting conclusions. The data thereon, however, are applicable to some extent to conditions of the proposed Gatun Lake, and indicate a considerable increase in supply other than that due to surface conditions. Figs. 48 to 53, inclusive, present graphically the conditions of rainfall, run-off, and retention which obtain in the basin above Alhajuela and over the entire drainage area above Gatun. The large quantity unaccounted for in minimum years, and the relatively small quantities "lost" in years of maximum rainfall, are, of course, what would be expected.

Hydrography of Gatun Lake and the Panama Canal.

Gatun Lake, which will be formed by the big earth dam across the Chagres River at Gatun, is one of the largest artificial bodies of water in the world, if not the largest. There is no similarly made body of its size in the Western Hemisphere, the nearest approach being the proposed Bear Lake Reservoir, in Utah, with a surface of 108 sq. miles, and it is probable that Egypt alone in the Eastern Hemisphere has a lake comparable either in area or capacity.

TABLE 47.—Areas and Volumes of Gatun Lake.

Contour. (Feet above mean sea level).	Area.		Volume.		
	Millions of square feet.	Square miles.	Millions of cubic feet.	Millions of gallons.	Acre-feet.
0	12	0.44	.....	.....	.....
10	385	13.81	1 982	14 825	45 499
20	813	29.17	7 972	59 637	183 031
30	1 426	51.14	19 169	143 385	440 062
40	1 971	70.69	36 152	270 419	829 942
50	2 529	90.71	58 651	438 707	1 346 434
60	3 099	111.17	86 791	649 198	1 992 450
70	3 670	131.65	120 638	902 375	2 769 476
80	4 274	153.32	160 361	1 199 507	3 681 401
85	4 555	163.38	183 136	1 369 859	4 204 228
90	4 835	173.44	205 911	1 540 211	4 727 056

At Elevation + 87 Gatun Lake has an area of 167.4 sq. miles, of which about 90 sq. miles are within the Canal Zone and the remainder in Panamanian territory. The shore line, not including islands, is about 1 010 miles in length, of which about two-thirds are without the Zone.

Table 48* is given for purposes of comparison. The reservoirs listed therein are either built or projected.

TABLE 48.

Name.	Capacity, in thousands of millions of cubic feet.	Name.	Capacity, in thousands of millions of cubic feet.
GATUN LAKE	205.91	New Croton Dam, N. Y.	7.85
Cataracte de Assouan, Egypt	130.65	Buena Vista Lake, Cal.	7.41
Au Sud de Philos, Egypt	126.40	Penai, India	6.90
Gebel Silsileh, Egypt	123.60	Hemet Valley, Cal.	6.00
Cataracte de Assouan, Egypt	94.40	Lake McMillan, N. Mex.	6.00
Kalabehele	90.40	Bhatgus, India	5.48
Gebel Silsileh, Egypt	84.50	Paragon, India	5.29
Swan Valley Lake, Idaho	65.30	Laramie River Dam, Wyo.	5.24
Ouady Rayan, Egypt	35.31	Lake Fife, India	4.91
Tonto Basin, Ariz.	32.00	Indian River, N. Y.	4.35
Belle Fourche, S. D.	9.35	Jarpen, India	2.69
Wachusett Dam, Mass.	8.40		

Total area, Gatun Lake Drainage Basin	1 320	sq. miles.
Land " (Lake surface El. 87 M. S. L.)	1 152.6	" "
Water " " " " 87 " " "	167.4	" "
Ratio of water to land surface	14.52	per cent.

The top of the upper miter-sill at the locks has an elevation of + 37.33 above mean sea level, and the bottom of Culebra Cut will be at Elevation + 40, referred to the same base. The largest vessel yet built is the *Olympic,* which, when loaded to full capacity of 60 000 tons, is said to have a draft of 37.5 ft. If the minimum lake level is assumed at Elevation + 80.33, this vessel would have about 5 ft. of water under her keel when passing over the miter-sills and more than 2 ft. when passing through Culebra Cut. The heaviest United States war vessel is the *Utah,* recently in commission, and she, when loaded, is said to have a draft of 28.5 ft.; that is, at the minimum lake level assumed above, the *Utah* could pass through Culebra Cut with at least 11 ft. of water under her keel.

A storage depth of 6.67 ft., or 80.04 in., is equivalent to 11.624 in. distributed over the land area at Elevation + 87; that is, a run-off of 11.624 in. from the total land area, would raise the surface of the lake 80.04 in. This quantity, therefore, is required to make good the loss in storage when the lake surface is drawn down to the minimum, or

* Abstracted in part from Wilson's "Manual of Irrigation Engineering," Schuyler's "Reservoirs for Irrigation, Water Power and Domestic Water Supply," Buckley's "The Irrigation Works of India," and Barrois' "*Les Irrigations en Egypte.*"

the reservoir will stand this much depletion without reaching a surface level below that taken as a minimum. Reducing to equivalents, 1 in. on the land surface equals 0.574 ft. of lake storage. It is evident that, after considering the several unavoidable and continuous losses, such as maintenance, power, seepage, etc., the remainder is the quantity available for lockage.

TABLE 49.

Condition.	Elevation.	Area, in square miles.	Storage capacity in cubic feet.
Extreme high water	+87.00	167.4	192 246 000 000
Extreme low water	+80.33	154.0	161 864 000 000
Difference	6.67	13.4	30 382 000 000

The continuous draft or loss to lake storage was placed by the original Consulting Board of Engineers as:

Power, light, etc.	275	cu. ft. per sec.
Leakage at gates	275	" " " "
Seepage and other losses	85	" " " "
	635	" " " "

and these figures have been used in the following investigation, although it has been stated that recent experiments on the Stoney gates in place have demonstrated that the leakage on that account was exceedingly small, and it is probable that the figures given above will be greatly in excess of actual conditions.

The total requirements for seepage, maintenance, leakage, etc., of 635 cu. ft. per sec. are equivalent to 3.793 cu. ft. per sec. per sq. mile of lake area, which, converted into inches, gives the results in Table 50 for the several months.

TABLE 50.

Month.	Equivalent Depth, in Inches.	
	Lake area.	Land area.
28-day	3.95	0.574
29-day	4.09	0.594
30-day	4.23	0.614
31-day	4.37	0.634

The rainfall on Gatun Lake surface has been estimated from the records at the stations in this area, and gaps in the record have been filled by using the observed ratios between the given stations and those in the vicinity where the records were continuous. After completing the monthly periods, from 1901 to 1911, inclusive, for the several stations to be used, the quantities from these stations were given proper weight in making up the final table.

TABLE 51.—Estimated Average Monthly Rainfall, in Inches, Gatun Lake Surface.

Lake Area at Elevation + 87 = 167 Sq. Miles.

Month.	1901.	1902.	1903.	1904.	1905.	1906.	1907.	1908.	1909.	1910.	Monthly mean.
May	10.62	11.73	12.49	10.67	22.72	11.09	8.56	18.14	11.83	12.96	13.08
June	11.27	6.33	9.88	12.63	9.18	11.77	14.30	13.15	13.51	11.83	11.38
July	12.51	11.73	14.15	11.35	9.56	14.62	9.76	9.24	11.42	18.08	12.24
August	16.57	6.78	12.82	9.61	19.24	14.55	13.36	17.85	9.35	13.16	13.33
September	15.91	11.02	12.32	13.06	9.97	13.54	10.36	9.62	11.17	15.65	12.26
October	16.22	14.56	11.44	8.49	17.78	13.50	18.04	12.64	15.52	15.29	14.35
November	31.15	12.50	23.43	19.93	13.94	20.65	12.62	16.87	34.84	22.76	20.87
December	7.16	5.78	16.15	6.08	9.33	16.00	4.20	6.08	21.12	20.18	11.21
January	19.07	1.68	6.38	7.51	1.22	2.33	2.14	6.83	3.92	0.59	5.17
February	0.63	0.28	1.59	1.39	1.39	1.80	0.97	4.15	5.44	1.86	1.95
March	2.12	0.94	2.11	0.34	1.58	2.47	1.93	3.04	7.31	1.51	2.34
April	5.22	1.98	12.01	1.56	6.99	1.01	2.59	6.34	6.79	4.57	4.91
Year	148.45	85.31	134.77	102.62	122.90	123.33	98.83	123.95	152.22	138.44	123.09

The yearly totals are for the run-off year beginning May 1st. The average monthly rainfall over Gatun Lake was computed from the following records:

Station.	Weight.
Monte Lirio	11
Trinidad	53
Bohio + Alhajuela ÷ 2	42
Bohio + Gatun ÷ 2	61

The lowest annual rainfall of record at Colon was 86.50 in., during the calendar year of 1884. The rainfall at Gamboa for the same year was 96 in.

The weights given are the numbers corresponding to the area of lake surface that appeared to be represented by this particular station. It may be noted that the average of the Bohio and Alhajuela rainfalls of record is very close to that of Gamboa, which is nearly in the center

of this district. It is realized that nothing is added to the accuracy of the records by the interpolations for supplying missing data, either in rainfall or run-off tables. On the other hand, no more reliable data are at hand, and the intention in extending these tables has been to afford a basis for comparison extending over a similar term of years in all cases. The tabulations thus made are given as the best available, in the judgment of the writer, and do not purport to be a record of conditions that have existed. From close personal observation for the past 4 years, the writer is of the opinion that the results which are based on these estimates are within reasonable limits of error.

Besides the rainfall and the unavoidable demands for water, there is a very considerable loss to storage due to evaporation. The records and a discussion of this phenomenon have been given previously. From

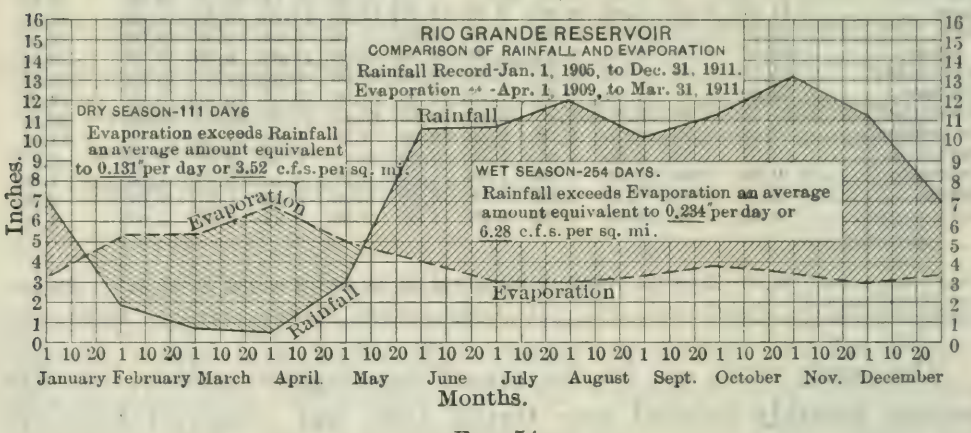

RIO GRANDE RESERVOIR
COMPARISON OF RAINFALL AND EVAPORATION
Rainfall Record-Jan. 1, 1905, to Dec. 31, 1911.
Evaporation -Apr. 1, 1909, to Mar. 31, 1911
DRY SEASON-111 DAYS
Evaporation exceeds Rainfall an average amount equivalent to 0.131" per day or 3.52 c.f.s. per sq. mi.
WET SEASON-254 DAYS.
Rainfall exceeds Evaporation an average amount equivalent to 0.234" per day or 6.28 c.f.s. per sq. mi.
Rainfall
Evaporation
Evaporation
Rainfall
Inches.
Months.
January February March April May June July August Sept. October Nov. December

FIG. 54.

these records, the diagrams on Plate XXXII and Figs. 53 to 55, inclusive, have been prepared to show the relations that exist between rainfall and evaporation on water surfaces on the Isthmus, and also to show an application to the surface of Gatun Lake. In many of the estimates there has been no allowance for the gain due to rainfall on the lake surface. The evaporation figure usually has been taken as equivalent to maximum conditions, and this has been applied flat throughout the year without reference to the rainfall. Fig. 53 and Plate XXXII give the actual conditions which existed during the calendar year, 1910, on Rio Grande and Brazos Brook Reservoirs. Fig. 54 shows the estimated evaporation from the surface of Gatun Lake had it been in existence during 1902-03, the year of minimum rainfall at Cristobal and that of estimated minimum on Gatun Lake.

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PLATE XXXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1260.
SAVILLE ON
HYDROLOGY OF THE
PANAMA CANAL.

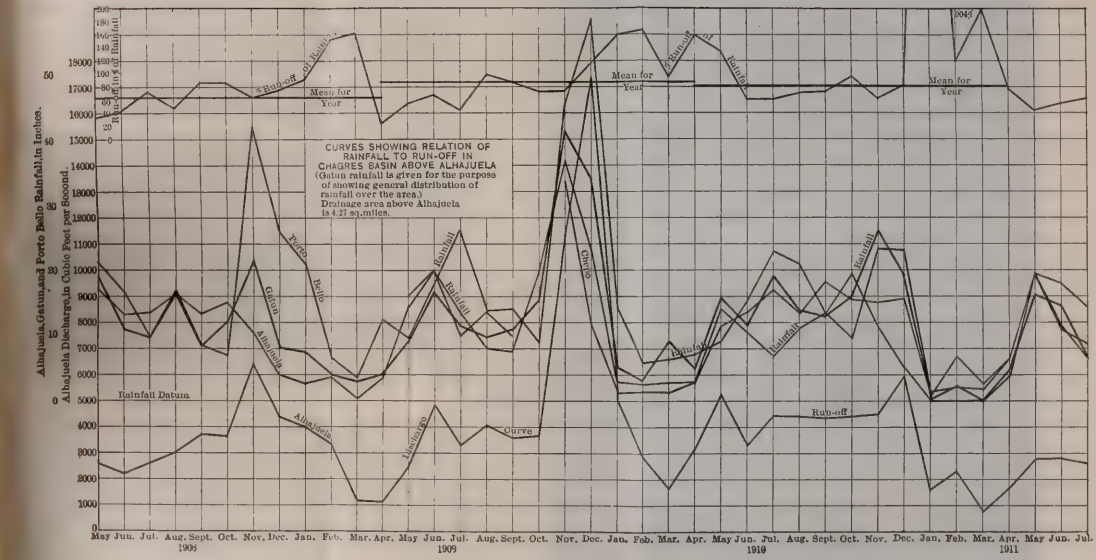

CURVES SHOWING RELATION OF RAINFALL TO RUN-OFF IN CHAGRES BASIN ABOVE ALHAJUELA
(Datun rainfall is given for the purpose of showing general distribution of rainfall over the area.)
Drainage area above Alhajuela is 4.21 sq. miles.
Alhajuela, Gatun, and Porto Bello Rainfall, in Inches.
Alhajuela Discharge, in Cubic Feet per Second
Rainfall Datum
% Run-off of Rainfall
Mean for Year
Porto Bello
Gatun
Alhajuela
Rainfall
Discharge
Curve
Run-off
May Jun. Jul. Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May Jun. Jul. Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May Jun. Jul.
1909
1910
1911

Fig. 55 shows the probable average conditions which would have existed on Gatun Lake during the past 10 years, had the lake been then in existence. These diagrams, together with that on Fig. 8, appear to indicate that the period of excessive daily evaporation is extremely limited, and that the apparent considerable losses to water surface during the so-called dry months are in reality greatly modified by the rainfall, which is especially persistent on the Gatun Lake area during this period. For investigations concerning water supply and storage conditions, the so-called mass-diagram presents a very satisfactory and simple solution of many problems, and a study of the character of the various curves brings out facts which might otherwise be overlooked. Desmond

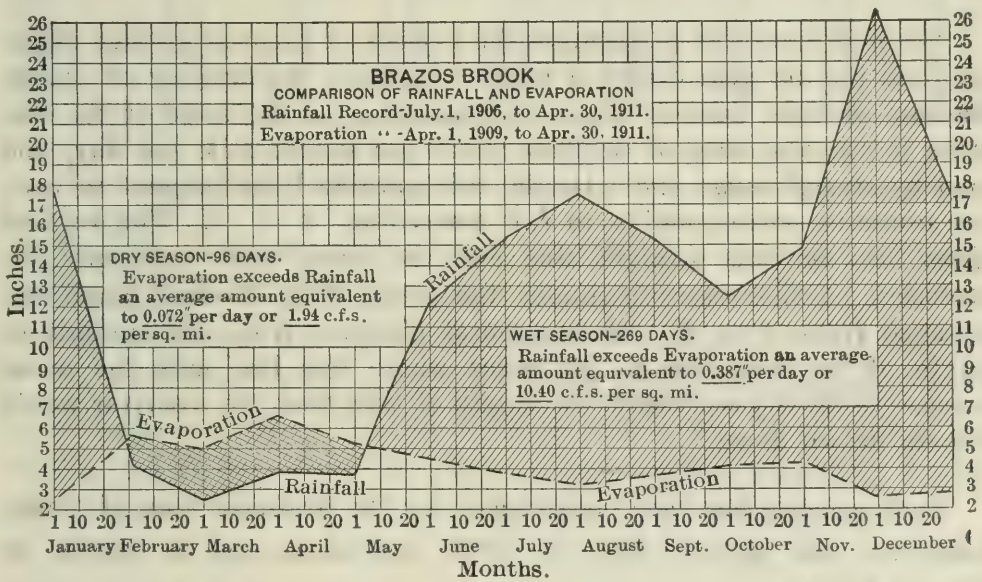

BRAZOS BROOK
COMPARISON OF RAINFALL AND EVAPORATION
Rainfall Record-July. 1, 1906, to Apr. 30, 1911.
Evaporation " -Apr. 1, 1909, to Apr. 30, 1911.
DRY SEASON-96 DAYS.
Evaporation exceeds Rainfall an average amount equivalent to 0.072" per day or 1.94 c.f.s. per sq. mi.
WET SEASON-269 DAYS.
Rainfall exceeds Evaporation an average amount equivalent to 0.387" per day or 10.40 c.f.s. per sq. mi.
Rainfall
Evaporation
Rainfall
Evaporation
Inches.
January February March April May June July August Sept. October Nov. December
Months.

FIG. 55.

FitzGerald, Past-President, Am. Soc. C. E., was probably the first to use this method in the United States, and it appears in his report on the "Capacity of the Sudbury River and Lake Cochituate Water-Sheds in Time of Drought," 1887. Mr. FitzGerald credits the use of this diagram to Mr. W. Rippl.* John R. Freeman, M. Am. Soc. C. E., has made the most extended use of this method in working up the probable yield of the Croton water-shed.† A very clear description is given therein of the construction and properties of this curve, and Theodore Horton, M. Am. Soc. C. E.,‡ has gone into the matter in

* *Minutes of Proceedings*, Inst. C. E., Vol. 71, p. 270.
† "Report on New York's Water Supply," Freeman, 1900, pp. 226–228.
‡ *Engineering Record*, July 31st, 1897.

considerable detail. Briefly, the mass-curve is the accumulated run-off of the drainage area (in this paper reduced to flow per square mile of land area) corrected for the rainfall and evaporation on the water surface, if a pond or reservoir is included in the area, and including the known constant withdrawals. Given this mass-curve of supply and the available storage capacity of the reservoir reduced to the same unit as the mass-curve, it is readily seen that, by projecting straight lines representing uniform rates of flow, it is possible to read directly from the diagram either the maximum draft that can be counted on during various seasons, or to find what will probably be the reservoir's depletion under various drafts. To quote Mr. Freeman:

"To find from this mass-curve the periods of greatest storage depletion and the dry year cycles of longest duration, we first lay off on the diagram straight lines at slopes corresponding on the scale of the diagram to various rates of uniform draft per square mile per day, and then by straight-edge and triangle, rule parallel lines tangent to various summits, corresponding to full reservoirs, * * *. The vertical space between this straight line and the curve now represents the depletion of the reservoir under the given rate of draft; and the distance from *A* to *E* [the horizontal distance from the summit or origin of the rate line to the point where this line cuts the mass-curve.—C. M. S.] represents the length of time that the reservoir would remain drawn down."

Such a curve has been constructed for the drainage area of Gatun Lake for the period, 1901 to 1911, inclusive, and is shown by Plate XXXIII.

In order that its construction may be clearly understood, the computations for the year 1902-03 are given in Table 52.

In Table 52: Column 2 is the run-off past Gatun (see Table 30). Column 3 is the estimated rainfall on the Lake (see Table 51). Column 4 is the mean evaporation for the past 2 years at Brazos Brook Reservoir, which, on account of its location, appeared to approximate most closely the conditions on Gatun Lake. Column 5 is the constant draft of 635 cu. ft. per sec., reduced to an equivalent in inches on the lake surface. Column 7 is the yield of the water surface of the lake in terms of the land surface; that is, Column 6 $\times \left(\frac{167.4}{1\ 152.6}\right) = 14.52$ per cent.

TABLE 52.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Months 1902–03.	Run-off or rainfall collected, in inches.	Rainfall on Gatun Lake, in inches.	Evaporation on Gatun Lake, in inches.	Constant Draft. Use and Loss; Power, Leakage, and Seepage, in inches.	Algebraic sums Columns 3+4+5, in inches.	Yield of water surface in terms of land surface 14.52%—Column 6, in inches.	Total yield, algebraic sum of Columns 2 + 7, in inches.	Accumulated yield, in inches.
Nov.....	9.14	12.50	2.44	4.23	5.83	0.85	9.99	132.25
Dec.....	4.25	5.78	2.56	4.37	—1.15	—0.17	4.08	136.33
Jan.....	2.29	1.68	5.46	4.37	—8.15	—1.18	1.11	137.44
Feb.....	1.31	0.28	4.89	3.95	—8.56	—1.24	0.07	137.51
Mar.....	0.90	0.94	6.51	4.37	—9.94	—1.44	—0.54	136.97
Apr.....	0.77	1.98	5.20	4.23	—7.45	—1.08	—0.31	136.66
May.....	2.14	12.49	4.45	4.37	3.67	0.53	2.67	139.33
June.....	3.48	9.88	3.66	4.23	1.99	0.29	3.77	143.10
July.....	5.70	14.15	3.03	4.37	6.75	0.98	6.68	149.78
Aug.....	7.75	12.82	3.47	4.37	4.98	0.72	8.47	158.25
Sept.....	7.79	12.32	3.99	4.23	4.10	0.59	8.38	166.63
Oct.....	7.89	11.44	4.17	4.37	2.90	0.42	8.31	174.94

The quantity for November in Column 9 is the sum of Column 8 from May, 1901.

The year, 1902-03, was taken as an example because its dry-season discharge was one of the lowest of the decade (1908 being about the same), and it was also the year of minimum rainfall in this period.

In order to compare quickly the given supply with assumed drafts at uniform rates, the vector diagram is added, with which it is a very simple matter to obtain the required information regarding storage, depletion, and length of time the lake will be below high water.

In his discussion on the "Water Supply for the Lock Canal at Panama,"* Col. H. F. Hodges states:

"With these assumptions [previously given.—C. M. S.], and taking into consideration the saving in water which can be effected by cross-filling, the total average consumption for one passage per day, *i. e.*, for two prisms of lift each day, is 44.2 cu. ft. per sec. Neglecting any advantage to be derived from cross-filling, it is 51.9 cu. ft. per sec."

In the following computation regarding the number of lockages possible per day, as far as water supply is concerned, it is taken as 51.9 cu. ft. per sec., or about 0.04503 cu. ft. per sec. per sq. mile of land area.

For converting cubic feet per second per square mile into inches

* *Transactions*, Am. Soc. C. E., Vol. LXVII. pp. 105–106.

depth on the tributary area, the following relations are convenient, 1 cu. ft. per sec. per sq. mile being equivalent to:

Period in days.	Inches.	Logarithms.
1	0.03719	8.5704262
28	1.04132	0.0174507
29	1.07851	0.0330214
30	1.11570	0.0476642
31	1.15289	0.0618292
365.25	13.58380	1.1330164

The factors in the construction of the rate diagram are given in Table 53.

TABLE 53.

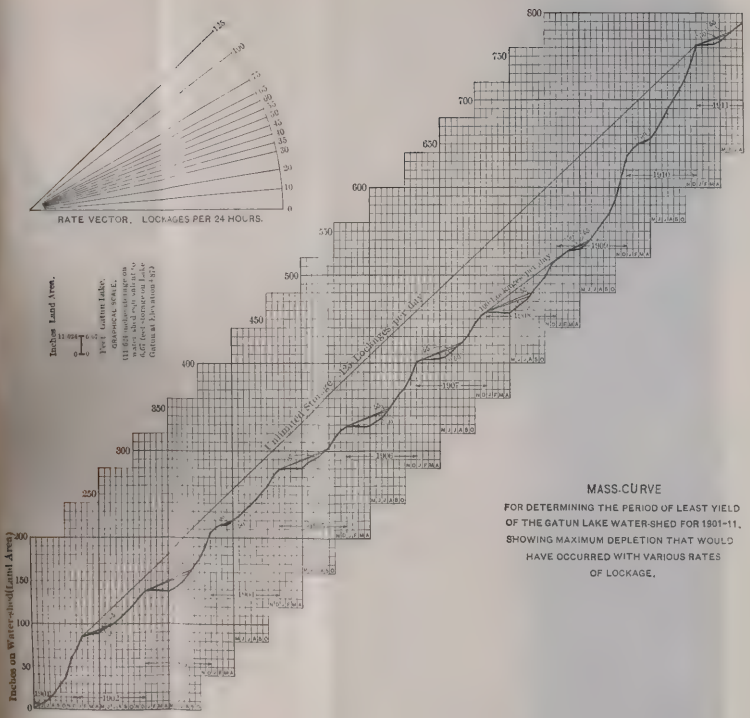
Number of lockages per day.	Cubic Feet Draft per Second.		Equivalent depth per year, in inches. Land area.
	Total.	Per square mile of land area.	
55.....	2854.5	2.476	33.635
50.....	2595.0	2.251	30.577
45.....	2335.5	2.026	27.519
40.....	2076.0	1.801	24.461
35.....	1816.5	1.576	21.403
10.....	519.0	0.4502	6.115

From inspection of Plate XXXIII it is evident that the critical periods, as far as storage is concerned, are to be found in 1903 and 1908. In order to study these conditions better, the diagram, Fig. 58, has been prepared, covering these same periods, but on a larger scale.

The construction of these diagrams is such that any measurement on the vertical scale represents inches depth per square mile of land area, and any measurement on the horizontal scale represents time in months. It is also to be remembered that the available lake storage is equivalent to 11.624 in. over the land surface. Therefore, direct measurement of the number of lockages* per day that can be made is possible, considering that the lake is full at the beginning of each dry season, and is not to be drawn below Elevation 80.33. The lake will be full at the beginning of the dry season if the line representing the required number of lockages does not in any year pass above the yield summit of the next following year.

* Wherever the word "lockage" is used in this paper, it means one complete passage of the canal; that is, the expenditure of water required for passing a vessel from one ocean up one slope and down the other to the ocean on the opposite side of the Isthmus.

PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1260.
SAVILLE ON
HYDROLOGY OF THE
PANAMA CANAL.



MASS-CURVE

FOR DETERMINING THE PERIOD OF LEAST YIELD OF THE GATUN LAKE WATER-SHED FOR 1901–11, SHOWING MAXIMUM DEPLETION THAT WOULD HAVE OCCURRED WITH VARIOUS RATES OF LOCKAGE.

In the case of the left-hand diagram of Fig. 58, the distance, 11.624 in., measured up from *F*, comes just above the line, *BC*, representing 55 lockages per day; and in the case of the right-hand diagram on Fig. 58, the distance up from *K* is a little above the line, *GH*, representing 55 lockages per day. Therefore, 55 lockages per day is

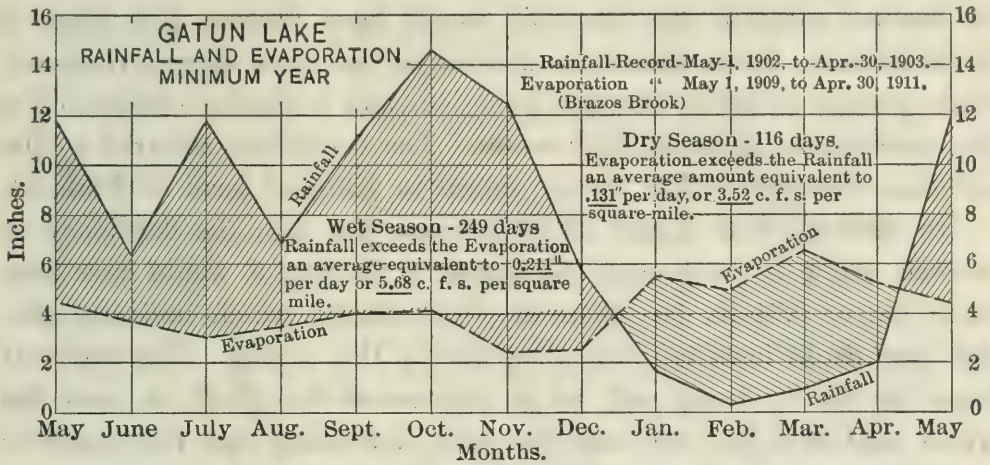

GATUN LAKE
RAINFALL AND EVAPORATION
MINIMUM YEAR
Rainfall Record-May 1, 1902, to Apr. 30, 1903.
Evaporation " May 1, 1909, to Apr. 30, 1911.
(Brazos Brook)
Dry Season - 116 days.
Evaporation exceeds the Rainfall an average amount equivalent to .131" per day, or 3.52 c. f. s. per square mile.
Wet Season - 249 days
Rainfall exceeds the Evaporation an average equivalent to 0.211" per day or 5.68 c. f. s. per square mile.
Rainfall
Evaporation
Inches.
May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May
Months.

FIG. 56.

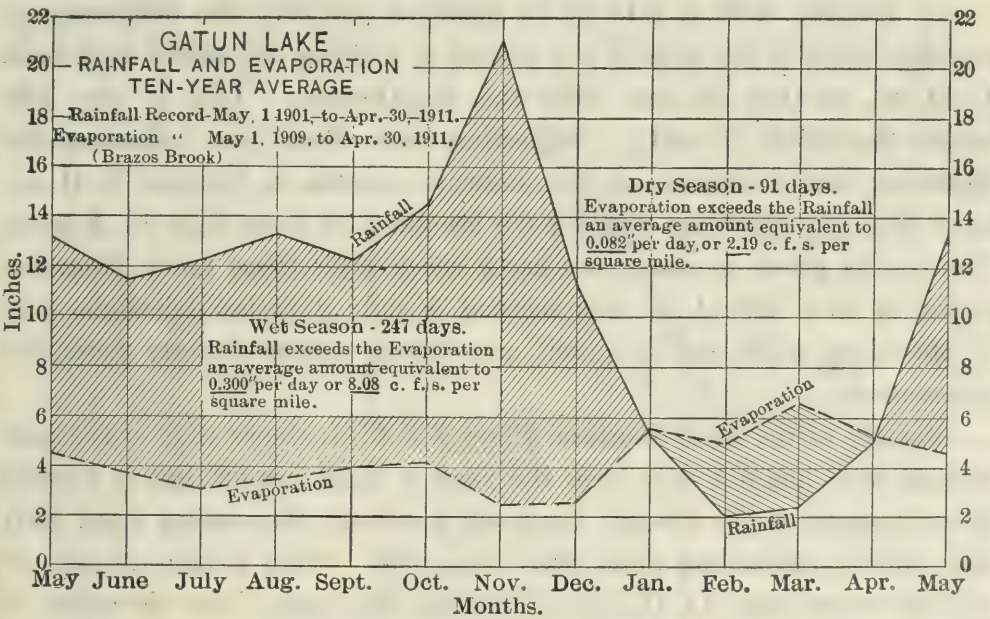

GATUN LAKE
RAINFALL AND EVAPORATION
TEN-YEAR AVERAGE
Rainfall Record-May, 1 1901, to Apr. 30, 1911.
Evaporation " May 1, 1909, to Apr. 30, 1911.
(Brazos Brook)
Dry Season - 91 days.
Evaporation exceeds the Rainfall an average amount equivalent to 0.082" per day, or 2.19 c. f. s. per square mile.
Wet Season - 247 days.
Rainfall exceeds the Evaporation an average amount equivalent to 0.300" per day or 8.08 c. f. s. per square mile.
Rainfall
Evaporation
Inches.
May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May
Months.

FIG. 57.

the maximum that can safely be counted on with present conditions of storage. On the horizontal scale each small subdivision represents a month. In the left-hand diagram, the line, *BC*, beginning January 1st, cuts the mass-curve a little after September 1st. With 55 lockages per day, the lake would begin to drop about January 1st, reach

its minimum elevation about May 1st, would then begin to fill, coming to high-water mark during the first part of September, and then wasting until the beginning of the next dry season. From the right-hand diagram it is seen that in 1908 similar conditions would have been experienced with the same number of lockages per day, but, on account of heavier rainfall, the reservoir would have become full about 2 weeks earlier. Lines drawn for greater or less rates show correspondingly greater or lesser storage, depletion, and a similar condition for the reservoir to be below high water. The conditions referred to the left-hand diagram of Fig. 58 are shown in different form in Table 54.

In 1902-03, with a rate of 55 lockages per day, the deficiency in storage, which is the sum of the figures in Column 5, would have been 10.88 in., or 0.74 in. less than the requirement, which indicates that this rate could have been supplied during this season. The reservoir began to fill in June, and, up to September 1st, 10.47 in. over the whole land area had run into the lake, indicating that full reservoir would have occurred some time during the first part of September.

In 1907-08, with a rate of 55 lockages per day, the deficiency in storage, which is the sum of the figures in Column 10, would have been 11.60 in., or 0.02 in. less than the requirement. This is also just inside the limit (11.624). Beginning with May, just following the depletion, and summing up the monthly surplus in Column 9, it appears that the reservoir would have become full some time in August. The results given in these two cases check with those taken from the curve, as they should, as this method is only a rather laborious way of obtaining what can be much more easily taken directly from the mass-curve.

A study of the mass-diagram, Plate XXXIII, shows that in the past decade, 1908 has not only been the year of least run-off, but is located at the bottom of the trough, the yield gradually decreasing from 1901 and rapidly increasing since May 1st, 1908. With a lake storage of a little more than 12 ft., or a limiting minimum lake elevation of + 75 instead of + 80.33, even during this period of low run-off, the lake would have been capable of caring for about 100 lockages per day. It appears, therefore, that under the present conditions of high-water elevation and storage it is possible to take care of 100 complete lockages per day without drawing the lake surface below Elevation + 75. Even with this rate of draft, the reservoir would have

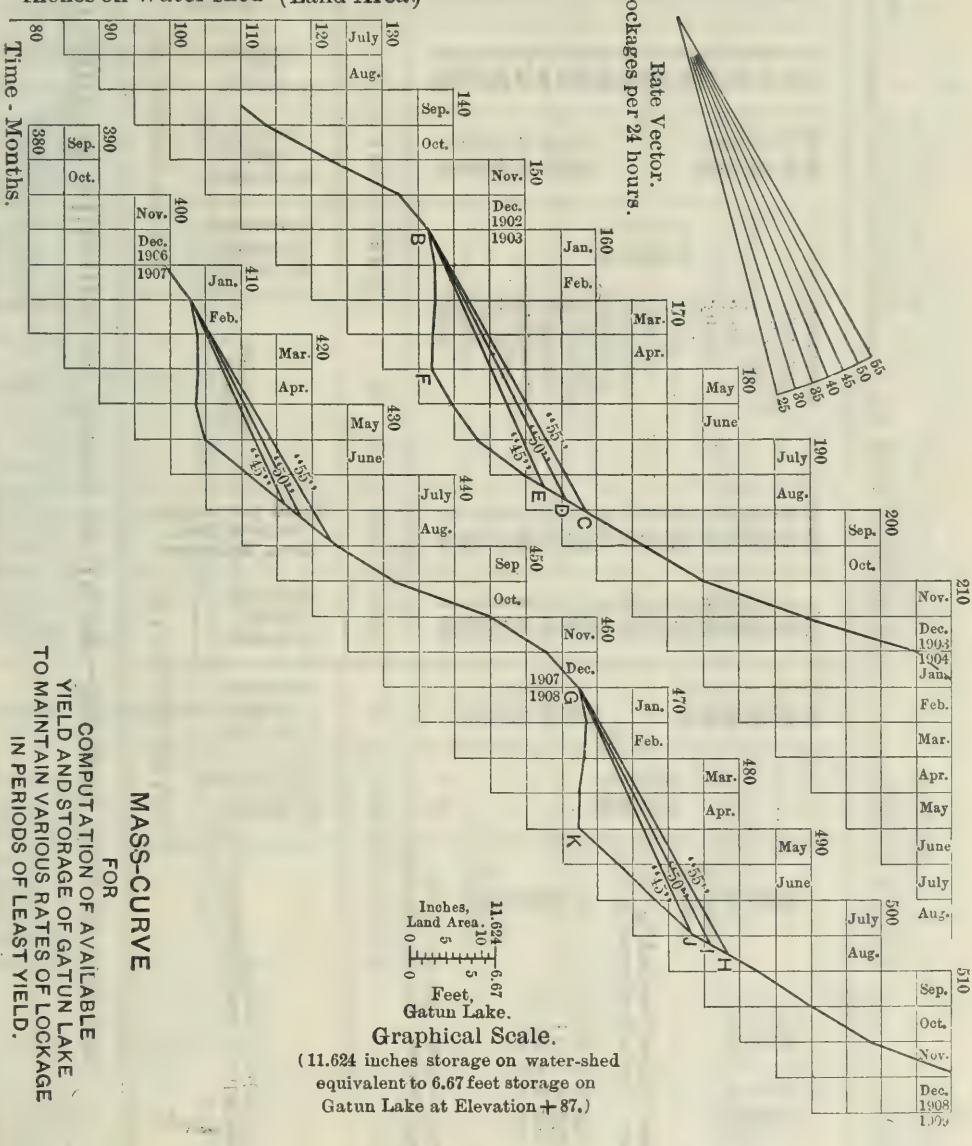

Inches on Water-shed (Land Area.)
Time - Months.
Rate Vector.
Lockages per 24 hours.
Inches, Land Area.
Feet, Gatun Lake.
Graphical Scale.
(11.624 inches storage on water-shed equivalent to 6.67 feet storage on Gatun Lake at Elevation + 87.)

FIG. 58.

MASS-CURVE
FOR
COMPUTATION OF AVAILABLE YIELD AND STORAGE OF GATUN LAKE TO MAINTAIN VARIOUS RATES OF LOCKAGE IN PERIODS OF LEAST YIELD.

TABLE 54.—Relation of Supply to Draft—Minimum Year of Run-off.

Month, 1902-1903.	Yield, per square mile of land area, in inches.	55 Lockages per Day.			Month, 1907-1908.	Yield per square mile of land area, in inches.	55 Lockages per Day.			Requirement for different periods and different rates of lockage.
		Requirement, in inches.	Surplus (waste), in inches.	Deficiency (storage), in inches.			Requirement, in inches.	Surplus (waste), in inches.	Deficiency (storage), in inches.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	
May	4.19	2.85	1.34	.....	May	1.58	2.85	.....	1.27	For 55 lockages per day, requirement, 2.476 cu. ft. per sec. per sq. mile.
June	4.12	2.75	1.37	.....	June	5.93	2.75	3.18	.....	
July	4.93	2.85	2.08	.....	July	6.57	2.85	3.72	.....	
August	4.65	2.85	1.80	.....	August	5.99	2.85	3.14	.....	
September	5.78	2.75	3.03	.....	September	7.92	2.75	5.17	.....	
October	9.26	2.85	6.41	.....	October	13.15	2.85	10.30	.....	
November	9.99	2.75	7.24	.....	November	8.72	2.75	5.97	.....	
December	4.08	2.85	1.23	.....	December	4.49	2.85	1.64	.....	
January	1.11	2.85	.....	1.74	January	0.62	2.85	.....	2.23	
February	0.07	2.58	.....	2.51	February	—0.31	2.67	.....	2.98	
March	—0.54	2.85	.....	3.39	March	—0.56	2.85	.....	3.41	
April	—0.31	2.75	.....	3.06	April	—0.23	2.75	.....	2.98	
May	2.67	2.85	.....	0.18	May	5.75	2.85	2.90	.....	
June	3.77	2.75	1.02	.....	June	5.01	2.75	2.26	.....	
July	6.68	2.85	3.83	.....	July	5.29	2.85	2.44	.....	
August	8.47	2.85	5.62	.....	August	9.21	2.85	6.36	.....	
September	8.31	2.75	5.56	.....	September	7.56	2.75	4.81	.....	
October	13.84	2.85	10.99	.....	October	8.34	2.85	5.49	.....	
November	16.19	2.75	13.44	.....	November	13.65	2.75	10.90	.....	

Period, in days.		Requirement, in inches.
28	1.041×2.476	2.577
29	1.078×2.476	2.667
30	1.115×2.476	2.761
31	1.152×2.476	2.852

For 50 lockages per day. Requirement, 2.251 cu. ft. per sec. per sq. mile.

Period, in days.		Requirement, in inches.
28	1.041×2.251	2.34
29	1.078×2.251	2.43
30	1.115×2.251	2.51
31	1.152×2.251	2.59

Note.—The yield in Columns 2 and 7 was obtained in the same way as the figures in Column 8 of Table 52. Some of the figures there given are repeated in this table.

TABLE 55.—Comparison of Rainfall for 1910-11 with Normal Rainfall for Years of Record.

Month.	Year 1910-11. Monthly, in inches.	Year 1910-11. Accumulated, in inches.	Normal for Years of Record. No. of Years.	Normal for Years of Record. Monthly, in inches.	Normal for Years of Record. Accumulated, in inches.
Cristobal Rainfall.					
November...	30.04	30.04	40	22.01	22.01
December...	15.20	45.24	40	12.58	34.59
January.....	0.99	46.23	41	4.05	38.64
February....	1.81	48.04	41	1.46	40.10
March.....	1.41	49.45	42	1.69	41.79
April	3.06	52.51	41	4.16	45.95
May	17.13	69.64	41	12.42	58.37
June	15.58	85.22	41	13.29	71.66
July	14.58	99.80	41	16.46	88.12
August.....	11.60	111.40	41	15.13	103.25
Gatun Rainfall.					
November...	26.06	26.06	6	24.03	24.03
December...	19.24	45.30	6	15.83	39.86
January.....	1.20	46.50	7	4.24	44.10
February....	2.19	48.69	7	2.41	46.51
March	1.80	50.49	7	3.25	49.76
April	6.38	56.87	7	4.03	53.79
May	19.14	76.01	7	15.93	69.72
June.....	14.72	90.73	6	13.61	83.33
July.....	6.66	97.39	6	12.51	95.84
August.....	7.91	105.30	6	14.73	110.57
Bohio Rainfall.					
November...	22.37	22.37	17	19.39	19.39
December....	21.30	43.67	16	11.25	30.64
January.....	0.32	43.99	16	6.00	36.64
February....	1.87	45.86	16	2.11	38.75
March.....	0.97	46.83	16	2.01	40.76
April	4.87	51.70	17	5.78	46.54
May.....	12.32	64.02	17	13.84	60.38
June.....	6.96	70.98	17	12.14	72.52
July.....	7.29	78.27	17	13.01	85.53
August.....	5.69	83.96	17	14.77	100.30
Gamboa Rainfall.					
November...	16.90	16.90	28	12.54	12.54
December...	13.11	30.01	28	7.31	19.85
January.....	0.11	30.12	26	1.91	21.76
February....	0.71	30.83	26	0.84	22.60
March.....	0.38	31.21	27	0.86	23.46
April	4.01	35.22	30	3.63	27.09
May.....	14.53	49.75	30	11.01	38.10
June	6.98	56.73	30	9.63	47.73
July.....	7.26	63.99	31	10.32	58.05
August.....	7.68	71.67	29	12.04	70.09

TABLE 55.—(*Continued.*)

Month.	Year 1910-11. Monthly, in inches.	Year 1910-11. Accumulated, in inches.	Normal for Years of Record. No. of Years.	Normal for Years of Record. Monthly, in inches.	Normal for Years of Record. Accumulated, in inches.
ALHAJUELA RAINFALL.					
November...	14.87	14.87	12	14.70	14.70
December ...	15.49	30.36	12	8.04	22.74
January	0.14	30.50	12	1.33	24.07
February....	2.29	32.79	12	0.83	24.90
March	0.01	32.80	12	0.71	25.61
April.....	4.88	37.68	12	3.47	29.08
May.....	16.11	53.79	11	12.69	41.77
June.....	10.53	64.32	12	12.92	54.69
July	8.84	73.16	13	13.47	68.16
August.....	10.79	83.95	13	13.10	81.26
CULEBRA RAINFALL.					
November...	10.81	10.81	22	12.52	12.52
December ...	11.86	22.67	20	8.19	20.71
January	0.02	22.69	21	1.83	22.54
February ...	0.74	23.43	21	0.57	23.11
March.....	0.06	23.49	21	0.71	23.82
April	4.89	28.38	21	3.83	27.65
May	14.86	43.24	21	11.31	38.96
June	4.25	47.49	20	8.92	47.88
July	5.95	53.44	20	9.50	57.38
August.....	8.36	61.80	20	10.46	67.84
TRINIDAD RAINFALL.					
November...	22.95	22.95	3	23.47	23.47
December ...	21.64	44.59	3	15.79	39.26
January.....	0.64	45.23	4	3.80	43.06
February....	2.00	47.23	4	3.28	46.34
March.....	2.33	49.56	4	3.89	50.23
April	3.33	52.89	4	5.74	55.97
May	18.20	71.09	4	14.65	70.62
June	8.79	79.88	4	10.90	81.52
July	3.63	83.51	4	8.94	90.46
August.....	8.17	91.68	4	12.18	102.64
MONTE LIRIO RAINFALL.					
November...	25.60	25.60	3	28.63	28.63
December ...	23.96	49.56	4	14.74	43.37
January.....	0.89	50.45	4	3.98	47.35
February....	2.41	52.86	4	4.60	51.95
March	1.41	54.27	4	4.64	56.59
April	5.27	59.54	4	5.77	62.36
May	19.70	79.24	4	15.84	78.20
June.....	11.43	90.67	4	13.64	91.84
July	10.58	101.25	4	14.76	106.60
August.	9.58	110.83	4	11.88	118.48
PORTO BELLO RAINFALL.					
November...	23.08	23.08	4	31.85	31.85
December....	22.89	45.97	4	30.07	61.92
January.....	0.84	46.81	4	9.64	71.56
February....	6.90	53.71	4	5.28	76.84
March.....	2.38	56.09	4	3.30	80.14
April	6.60	62.69	4	8.07	88.21
May	19.25	81.94	4	14.82	103.03
June.....	18.04	99.98	4	17.00	120.03
July	14.48	114.46	4	18.40	138.43
August.....	22.56	137.02	4	18.52	156.95

filled by January 1st, 1909. The general trend of the mass-curve indicates that the rate of 100 lockages per day is the reasonable maximum capacity of the canal, as far as water supply is concerned. Below this rate the storage lies in the present lake, provided Elevation + 75 can be allowed for the minimum surface elevation. For more than 100 lockages per day, the storage requirement increases so rapidly that it could be developed only at a considerable expense.

With practically unlimited storage during the past decade, the run-off would have been sufficient to have provided for about 125 lockages per day. To do this would probably mean additional storage reservoirs on tributary streams to conserve water which, under present conditions, will waste during the rainy months.

The rainfall on the Isthmus since January 1st, 1911, has been light, and below normal at many stations. The run-off for the tributary area has been affected, and the monthly flow past Gatun is the lowest of record (either by measurement or estimation) since 1890. Comparisons of the rainfall records in the drainage area for this period with the normals for the years of record are given in Table 55.

Table 56 is a comparison of the run-off at Alhajuela and Gatun for the present period, and the average of the monthly records of these stations for 1890-1911.

TABLE 56.

Month.	Alhajuela Station. Mean run-off, in inches.				Gatun Station. Mean run-off, in inches.			
	1911.	Accumulated.	1890-1911, inclusive.	Accumulated.	1911.	Accumulated.	1890-1911, inclusive.	Accumulated.
January...	4.427	4.427	6.214	6.214	3.170	3.170	5.358	5.358
February..	5.634	10.061	3.126	9.340	2.364	5.534	2.096	7.454
March.....	2.375	12.436	2.079	11.419	1.257	6.791	1.417	8.871
April.....	4.418	16.854	2.967	14.386	1.752	8.543	1.991	10.862
May.....	7.528	24.382	6.122	20.508	5.200	13.743	5.396	16.258
June.....	7.531	31.913	6.216	26.724	5.489	19.232	5.627	21.885
July.....	7.044	38.957	7.924	34.648	6.030	25.262	7.467	29.352
August....	6.364	45.320	8.157	42.805	3.954	29.216	8.181	37.533
September.	4.786	50.107	7.529	50.334	3.805	33.021	7.864	45.397

The run-off past Gatun is affected constantly by two conditions which do not obtain in the basin above Alhajuela:

1.—Extensive swamp areas with a cover of vegetation that require large quantities of water; and

2.—Evaporation from Gatun Lake surface, which was not experienced in former years.

In addition to the foregoing, the greatest factor contributing to the large proportional quantity of run-off per square mile above Alhajuela, compared with that of the basin as a whole, probably is to be found in the greater precipitation on the former area. This condition is found in Tables 44 and 45, from which it appears that the annual average rainfall from the whole basin for 10 years was 125.04 in.; for the portion below Alhajuela, 118.90 in.; and for the portion above that station, 136.36 in. In other words, the rainfall on the area above Alhajuela is about 9% greater than the average on the basin as a whole, and 14.5% greater than that on the basin below that station.

TABLE 57.

Month.	Mean lake elevation.	Mean area of lake surface, in square miles.	Evaporation, Brazos Brook, in inches.	Evaporation in cubic feet per second, per square mile.	Equivalent evaporation for actual lake surface, in cubic feet per second.	Equivalent evaporation for proposed lake area (167.4), in cubic feet per second.
	(1)	(2)	(3)	(4)	(5)	(6)
January...	14.09	19.60	6.29	5.46	107	913
February..	13.71	18.90	5.12	4.92	93	822
March. ...	12.55	17.30	6.87	5.96	103	998
April.....	13.12	18.10	4.94	4.43	80	740
May	15.58	21.70	3.29	2.85	62	478
June.....	15.58	22.10	2.92	2.62	58	438
July.....	15.92	22.30	4.36	3.78	84	634
August....	14.50	20.50	4.07	3.53	72	592
September	14.48	20.45	4.10	3.56	73	596

The figures in Column 5 of Table 57 should be added to the monthly discharge of Table 28, since April, 1910, in order to compare these 3 months with the discharge of previous months of the same name.

Table 58 is given to show what would have been the effect on the lake and its usefulness had it been in operation during the period since January 1st, 1911.

Table 58 shows that, even with the low rainfall and similar conditions of run-off since January 1st, 1911, the lake would have handled 55 lockages per day, and the surface then would have been above the limit by 2.38 ft., equivalent to 4.15 in. in land surface.

TABLE 58.—Fifty-five Lockages per Day.

Month, 1910–11.	Yield, corrected for evaporation on lake area, in inches.	Requirement, in inches.	Surplus.	Deficiency.
November	16.98	2.75	14.23	.....
December	21.01	2.85	18.16	.....
January	1.80	2.85	.....	1.05
February	1.38	2.58	.....	1.20
March	0.23	2.85	.....	2.62
April	0.15	2.75	.....	2.60
May	6.51	2.85	3.66	.....
June	5.92	2.75	3.17	.....
July	5.75	2.85	2.90	.....
August	3.90	2.85	1.05	.....
September	3.62	2.75	0.87	.....

From that which has preceded, therefore, it appears that the water supply of Gatun Lake is sufficient for any probable demands of commercial or other navigation; and the use of the canal is limited only by the capacity of the locks and the requirements of practical operation.

DISCUSSION

Gen. Abbot.

HENRY L. ABBOT, ESQ.* (by letter).—Mr. Saville has made an interesting and valuable contribution to the technical literature of the Panama Canal. He has had a prominent part in directing the meteorological and hydraulic investigations, and the results set forth in detail in this paper will command general attention. Such records, covering one more of the greatest known floods of the Chagres, would have been of great service in the studies which determined the character of construction to be adopted—whether with lake and locks, or at sea level. The contours of the bed of the lake and the area of the watershed were then known only approximately, and the statistics as to rainfall and evaporation were much less complete than at present. It is pleasing to note that the new data confirm the conclusions and plan finally adopted, and set at rest any doubts as to the sufficiency of the water supply for all the operating needs of the Canal; and this without resort to the expedient of a reservoir above Alhajuela, to conserve the excess of flow in the rainy season which otherwise might go in part to waste over the spillway.

It may be interesting to note that such a reservoir formed an essential element in the plan adopted by the New French Company, in order to augment the water supply afforded by the lake to be formed by the proposed dam at Bohio, a lake storing a much smaller volume than Lake Gatun, but which financial reasons compelled the Company to adopt, as the Old French Company had already excavated a sea-level canal to the vicinity of Bohio. There is a favorable site for a masonry dam a short distance above Alhajuela, of which the cost was estimated at about $2 500 000, and Mr. Saville's surveys and observations in the Upper Chagres Basin have confirmed the opinion that a large volume of water may be stored there, if it is ever found to be desirable.

There are two reasons why such an addition to the present plan may be found to be advisable ultimately. The sudden and violent freshets which characterize the regimen of the Chagres will attain the canal route near Gamboa, where the narrow arm of Lake Gatun turns nearly at right angles to the direction followed by shipping. The channel here, for about four miles, is only 500 ft. wide, and it is not impossible that these sudden but brief rushes of water may prove to be more or less annoying. The largest freshets of record at Gamboa have carried maximum volumes ranging from about 65 000 to 79 000 cu. ft. per sec., with an average flow during 24 hours falling but little below 60 000 cu. ft. per sec. Their annual frequency, as well as the amount of rainfall on the Isthmus, seems to be subject to a progressive variation, ranging at Gamboa during the last 40 years from about 10 to about 35 annually, with a corresponding duration, including large

*Brigadier-General, U. S. A. (Retired).

and small freshets, ranging from about 75 to about 650 hours. A dam above Alhajuela would regulate this variable flow perfectly and eliminate all objectionable currents from the route for shipping. The second reason why an Alhajuela dam may become desirable is that, in these days of water-power development through the agency of electricity, it may be found convenient to augment the quantity of water available on the Isthmus by conserving the flow which otherwise, at such times, might run to waste over the spillway. Whether there will be such a demand can best be decided when the Canal is opened to traffic. Gen. Abbot.

W. E. Fuller, M. Am. Soc. C. E. (by letter).—This paper contains interesting information on the flood discharge of the Chagres River. The flood at Alhajuela, on December 26th, 1909, with a maximum rate of discharge of 170 000 cu. ft. per sec., on a catchment area of 427 sq. miles, ranks high among the floods of which the writer has record. Mr. Fuller.

By applying the writer's formula,

$$Q = KA^{0.8}\,(1 + 2\,A^{-0.3}),$$

to this flood, a value of $K = 1\,010$ is obtained. The greatest floods of the eastern portion of the United States give values of $K = 300$ to 360. For floods giving values of K similar to that for the Chagres at Alhajuela, the writer has only found records on a few rivers in India and in Saxony. It seems probable from the limited data that similar floods occur on some rivers on the Pacific Coast, but no records of such maximum rates are available.

It is interesting to note that in all three floods mentioned by the author the maximum rate of discharge was greater at Alhajuela (catchment area, 427 sq. miles) than at either Bohio (catchment area, 779 sq. miles), or Gatun (1 320 sq. miles). It would be interesting to know whether other floods indicate similar results, and if so, whether the conditions of temporary storage below Alhajuela satisfactorily account for the difference.

Although the data are not very complete, those for the floods at Gamboa give an indication of the frequency of flood occurrence. An application of the writer's formula,

$$Q\ (\text{Max.}) = C\,A^{0.8}\,(1 + 2\,A^{-0.3})\,(1 + 0.8 \log.\ T),$$

to the data given by the author, and by General Abbot in his discussion, gives a probable value of C for Gamboa of 170 to 200. The floods at Bohio and Gatun indicate values somewhat smaller, and those at Alhajuela indicate a value of from 350 to 450.

Values of C, for streams in the eastern part of the United States, seldom exceed 100, with 65 as an average; it appears, therefore, that this tropical river produces floods from three to five times as great as those rivers.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1261

A SUGGESTED IMPROVEMENT IN BUILDING WATER-BOUND MACADAM ROADS.*

By J. L. Meem, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. F. G. Frink, C. H. Sweetser, W. W. Crosby, J. C. Meem, Arthur H. Blanchard, and J. L. Meem.

In this era of vast road building throughout the country, it has seemed to the writer that there is too much variance in the specifications, especially in those for water-bound macadam roads, and this paper describes what is believed to be an improvement in such road construction.

By way of explanation, it may be stated that water-bound roads are those in which the only binder is water, and this is used very plentifully in the process of building the road. There is no other foundation than the rolled earth; the standard depth of loose rock ordinarily used is 9 in., and when this has been thoroughly rolled, a theoretical depth of 6 in. of finished roadway is obtained. The ordinarily accepted specification for placing this rock calls for 6 in. of No. 1 stone (the largest size) varying from 1½ to 3 in. in diameter; then 2½ in. of No. 2 stone, varying from ½ to 1½ in. in diameter, and the top course is finished off with stone ½ in. and less in diameter, usually called "screenings," including the material resulting from the dust of fracture. Assuming, then, a depth of loose rock of 9 in. as a standard, the writer describes herein a method of proportioning this rock which he has used successfully and is using at the present time on an 8-mile section of the Memphis-Bristol Highway, in Tennessee.

* Presented at the meeting of February 19th, 1913.

All road builders, both engineers and contractors, will probably agree that the difficult and essential thing to obtain in a water-bound road is the bond, and the method herein described attains this result at probably less cost than any other, and certainly at very much less cost than any the writer has ever tried.

The method consists in separating the screenings into two sizes, one size being from ½ to ¼ in. and the other the dust of fracture, and putting these two sizes on the road in separate operations, and the writer wishes to suggest the adoption of the following specifications for building a water-bound macadam road, both as to the method of proportioning the rock and as to sprinkling:

(*a*) The sub-grade shall be thoroughly rolled until it is firm and compact, with the proper crown of ¾ in. per ft.

(*b*) On this shall be placed 6 in. of No. 1 grade rock, ranging in size from 1½ to 3 in. This shall be spread uniformly and rolled dry until the rock does not creep before the roller, or creeps just enough to set it in place without crushing.

(*c*) Then 2 in. of stone of No. 2 grade, ranging in size from ½ to 1½ in., shall be spread uniformly, sprinkled, and rolled thoroughly into the voids of the No. 1 grade.

(*d*) Then ¾ in. of the coarse material from the screenings, consisting of sizes from ¼ to ½ in., shall be spread uniformly, sprinkled, and rolled thoroughly into the voids of the No. 2 grade.

(*e*) Finally, ¼ in. of the dust from the screenings shall be spread (preferably by hand from shovels), thoroughly soaked, and rolled to a finish.

This method gives a surface which is virtually impervious to water, and is less affected by automobile traffic than any with which the writer has had experience; in fact, it has been a matter of great surprise to him to see how well roads built by this method withstand automobile traffic without "raveling" or showing signs of deterioration.

From the contractor's point of view, as an economical method of building macadam roads, the writer finds that on the Memphis-Bristol Highway he is able to care for 100 cu. yd. of rock per day with one 10-ton roller, where previously, before the change was made, there was some difficulty in rolling properly one-half this quantity and getting the proper bond therein, in spite of the fact that it requires

one more operation to put on the extra course of stone. This is very much more than balanced by the quicker method of obtaining the required bond, and the final results as stated are so much more satisfactory than those obtained by using only three grades of rock and not separating the dust from the screenings that it would seem to be a material improvement in the method of road building, and should appeal to both engineers and contractors. It is only fair to say that the specification under which this section of the Memphis-Bristol Highway is being built called for the placing of the macadam in three operations, as originally stated, but that the change has been approved by the engineer in charge who now highly endorses the method.

This work requires a standard screen with a "dust jacket" (¼-in. perforations) over the ⅝-in. screen.

The writer, who is a member of the contracting firm which is building this section of the road, is interested both from the standpoint of the engineer and the contractor. He has never before seen this method advocated publicly, and invites discussion and criticism from other road builders.

DISCUSSION

F. G. Frink, Assoc. M. Am. Soc. C. E. (by letter).—It is hoped that Mr. Meem will publish such information in regard to topography, climate, sub-grade, kind of stone, expected traffic, etc., as will show the range of adaptation of the design presented. In this way costly experiments may be avoided. Mr. Frink.

The writer's experience leads to the conclusion that the "variance in specifications," deplored by the author, is really one of the most encouraging features in the development of road building, as it implies a recognition of the widely differing conditions to be met, The use of a design, because it has been a success in some other locality, may result in loss of time and money.

For example, experience teaches that a road constructed with such a base as the one cited would probably fail in a section where, by porosity of the soil and seepage from higher land, the sub-grade was saturated for extended periods. In this case the base must be rendered as nearly impervious as possible by using fine material at the bottom to fill the voids.

Bearing in mind Macadam's insistence on the use of small fragments, the writer would suggest the query: Is a road ever the better for having large stone, say, greater than 1½ in., in any part of its section?

C. H. Sweetser, M. Am. Soc. C. E. (by letter).—The method of separating the screenings into two sizes, as suggested by Mr. Meem, and used successfully on the Memphis-Bristol Highway, in Tennessee, seems to be an improvement over the usual specification for the application of the binder course. The writer has obtained a quicker bond by applying the screenings in small quantities until the required bond was obtained. When the dust of fracture is applied, together with the coarse material in the screenings, the dust when wet seems to "ball" up and prevent the coarse material from filling quickly the voids in the macadam. From the experience on the Memphis-Bristol Highway, the suggested improvement would seem to hasten the obtaining of a proper bond. Mr. Sweetser.

A few words on water-bound macadam and the automobile may not be out of place. Engineers should hesitate, and consider local conditions very carefully, before specifying the construction of water-bound macadam highways. With the improvement of the highway, automobile traffic is sure to increase, and it soon becomes necessary to construct a surface which will withstand this traffic. In the majority of cases the most economical time to do this is when the improvement is first made. Mr. Meem states that the suggested improvement is

Mr. Sweetser. "less affected by automobile traffic." Why not construct at once, that it will not be affected at all?

In spite of knowledge gained through the experience of road builders, both in the United States and Europe, there are miles of water-bound macadam roads being built in America where some other form of pavement should have been specified.

Mr. Crosby. W. W. Crosby, M. Am. Soc. C. E. (by letter).—The author refers to the "bond" of the road, apparently meaning the cemented condition brought about by the action of water on the stone dust. If this deduction is correct, the writer cannot agree with one of the author's premises, namely, that such a "bond" is "the difficult and essential thing to obtain in a water-bound road."

If, however, the "bond" is, as the writer understands the term, the interlocking of the angular particles of stone with each other, supplemented by the cementing of the fine material, then the writer is prepared to agree with Mr. Meem to a certain extent. The qualification he has in mind is that such a bond, while always essential, is only difficult to secure with proper stone where the sub-grade for any reason is inclined to churn up into the macadam stone and by its intrusion prevent their proper compaction. Such a bond, of course, costs something to secure by rolling, but this does not seem to the writer to be a real "difficulty."

The method of construction proposed by Mr. Meem has been tried by the writer on various occasions, but without his becoming favorable to it. In one or two instances, where "granolithic" or "grit" (the chips between the $\frac{1}{4}$- and $\frac{1}{2}$-in. screens) was in demand for other purposes, the crusher product was robbed of a part of this portion and the remainder was used as described by the author in two applications, without other benefit than securing in this way some "grit" for another use.

It seems to the writer that probably the superiority of results, stated by Mr. Meem as secured by his method, comes from the application of the screenings in two relatively thin layers with thorough rolling between them.

Too many roads are built where, after rolling the second course (or "No. 2" stone), the screenings are applied heavily in one coat an inch or more thick, and rolled and watered until the surface seems to be cemented. Such roads seldom wear well, raveling shortly and continually. The cementing thus secured is superficial, and the macadam beneath is porous. If, however, the screenings are applied in thin layers, and each layer is flushed and rolled into the voids of the "No. 2's," then the macadam will be more dense and less likely to ravel. In the writer's opinion, with the latter method, it makes very little difference how the screenings then are made up as to sizes

unless they come from limestone. In the latter case a sufficient quantity of coarse particles or non-cementitious material must be added, by mixing or by separate application, to offset the extreme tendency of the fine limestone material, by forming a paste on the surface, to prevent the proper filling of the voids in the macadam. Mr. Crosby.

The writer's specifications, in this particular, have been as follows:

"THIRD COURSE.

"Material.

"173.—The third course of the macadam construction shall consist of trap rock screenings varying in size from dust to 1-inch pieces. Other material than trap rock screenings may be used if approved by the Engineer. Limestone screenings shall not be used with a limestone second course, unless approved in writing by the Engineer, or unless clean sand, approved by the Engineer, is used in substitution under the direction of the Engineer, for at least 50% of the limestone screenings.

"Spreading.

"174.—After the second course of No. 2 stone has been rolled and completed as above described, the screenings shall be spread, but in no case shall screenings be used until the second course has been thoroughly rolled and compacted. The screenings shall be spread dry, with shovels from piles alongside the road or from dumping boards; but in no case shall the screenings be dumped directly on the second course. The quantity of screenings used shall be such as will just cover the second course.

"Watering and Rolling.

"175.—After the screenings are spread they shall be first rolled dry and then sprinkled with water from a properly constructed sprinkling cart, and then re-rolled with a steam roller weighing not less than ten tons. The amount of water necessary shall be determined by the Engineer. The rolling shall begin at the sides and continue until the surface is hard and smooth and shows no perceptible tracks from vehicles passing over it.

"176.—If, after rolling the screenings, the No. 2 stone appears at the surface, additional screenings shall be used in such places.

"177.—The rolling and watering shall continue until the water flushes to the surface. The rolling is to extend over the whole width of the road, including the shoulders."

The closing sentence of Paragraph 174 should be noted, and this, followed by Paragraph 176, illustrates the point above made.

The writer thinks he sees one danger in the method proposed by Mr. Meem. It is that, by first partly filling the voids in the surface of the "No. 2's" through the application of the grit and by then completely filling the remaining voids in the surface by the application of the finer sizes, the penetration of the finer sizes into the voids in the body of the macadam will be obstructed or prevented. It is the

Mr. Crosby. writer's experience that the best macadam will have a maximum density, not only on or near its surface, but also well down into the macadam itself.

Mr. J. C. Meem. J. C. MEEM, M. AM. SOC. C. E. (by letter).—It would appear to the writer that a road built under the author's specifications would undoubtedly have good wearing qualities if it did not become too dry. It is suggested, therefore, that a frequent sprinkling or application of oil would insure its lasting quality in the same ratio that its original value might bear to that of a road built by the alternative method.

The presentation of this paper reminds the writer of a piece of macadam road which the author constructed on a private estate in Virginia. About 2 miles of ordinary country road were macadamized, and a highway bridge was built over a tributary of the Shenandoah River. The macadam work consisted of a 12-ft. roadway, which was "metalled" according to the specifications of this so-called improved method, with berms of earth 3 ft. wide on each side, and with under-drains outside of these, as noted later. This road is wearing very well, after 2 years of such traffic as country roads ordinarily get from heavy farm wagons and occasional automobiles; and, although no comparative values are obtainable, its good condition is believed to be due partly to the fact that under-drains, in a trench 12 in. wide and 2 ft. deep, and surrounded with broken stone, were laid to a uniform grade on each side, giving perfect drainage to the highway. The rock used was the native blue limestone, quarried and crushed on the estate.

In connection with this work, the writer designed a reinforced concrete highway bridge, which was built by the author. The question of surfacing the roadway of the bridge was an important one. It was not deemed advisable to carry the macadam over the roadway, as it was thought that its tendency to dry out and disintegrate would be much greater than in an ordinary road in contact with earth. The following method was finally adopted: The upper or compression side of the concrete was troweled to a very compact and smooth surface, and, on this, there was placed about 1 in. of lean cement mortar, over which, in turn, from $\frac{1}{2}$ to $\frac{3}{4}$ in. of sand was sprinkled before the mortar had set. The bridge has now been in use about 2 years, and this surface is apparently giving satisfaction. As this method of surfacing was in the nature of an experiment, the writer will be greatly interested in observing from time to time the wearing qualities which it develops.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—Two properties of wearing surfaces which require consideration in the design of pavements are stability and density. The requisite degree of stability and density depends in part on the conditions to which the road or pave-

Mr. Blanchard.

ment is to be subjected. The author has outlined a method for securing a certain degree of stability and density in the construction of the so-called water-bound macadam road.

The efficacy of the proposed method depends primarily on the traffic to which the road is to be subjected. If the traffic consists of horse-drawn vehicles not exceeding a certain number per day, the use of the type of construction advocated should result in a serviceable road. If, however, many high-speed motor cars use the highway thus built, the surface will disintegrate rapidly, as is the case with macadam roads built with a wearing surface of so-called crusher-run stone (passing over a ½-in. screen and through a 1½-in. screen), the surface of which is finished with a light coat of screenings, well rolled and watered. The difference between the former and the latter is in the arrangement of the aggregate, rather than in its composition, as is seen by a consideration of the following mechanical analysis of the wearing surface of the latter type.

Broken stone passing			⅛	-in.	screen	.....	1.0	per	cent.
"	"	"	¼	"	"	.....	2.5	"	"
"	"	"	½	"	"	.....	30.8	"	"
"	"	"	¾	"	"	.....	34.2	"	"
"	"	"	1	"	"	.....	23.4	"	"
"	"	"	1¼	"	"	.....	8.1	"	"

The analysis of the author's wearing surface of 2 in. would likewise show certain percentages passing screens from ¼ in. to 1¼ in. In both cases, stone from ½ in. to 1¼ in. in diameter will usually constitute a large percentage of the top inch of the surface.

The speaker's investigations and practice have led to the conclusion that the most satisfactory type of macadam road to withstand high-speed motor-car traffic is one having a surface of broken stone, varying from 1 in. to 2½ in. in longest dimensions, which stone has been thoroughly rolled and puddled. The success attendant on the use of bituminous surfaces on macadam roads in England and France is due in part to the adoption of this type of construction. Not only does the use of the large broken stone in the surface give stability to its component integral parts, but it simplifies one essential detail of the construction of bituminous surfaces on macadam roads, namely, the cleaning of the macadam surface.

With the addition of bituminous cement, the status of the pavement, as one adaptable for use on highways subjected to motor-car traffic, is changed. The method of construction in which several courses of varying sizes of stone are used formed the basis of many

Mr. Blanchard.

of the patents covering the construction of bituminous concrete pavements granted by the United States Government during the period from 1869 to 1873. Bituminous concrete pavements of this type have given excellent results, in the United States and also abroad. At present, several foreign companies, particularly in England, are using this method of construction. The work of three of these companies is referred to in the following descriptions:

The Quarrite-Bitulithic Company of London imports from its Scotland plant bituminous-coated broken stone of various sizes for the construction of bituminous concrete pavements by the layer method. The sizes used for the courses are as follows: first course, 2 in. of 1-in. to 1¾-in. stone, rolled, with a scattering of ⅛ to ⅜-in. chips; second course, 1 in. of stone, varying from ¼ to ¾ in. in longest dimensions, rolled; third course, a thin layer of tar-coated screenings, thoroughly rolled.

The Tarmac Company of London for many years has been mixing broken slag of various sizes with bituminous cement at its plant in Wolverhampton, and shipping from there to various points in England. The construction of one pavement of this type which came under the speaker's observation is as follows: This road was the main shore boulevard at Brighton-on-Sea, on the south coast of England. On a well-compacted gravel foundation was spread a scattering of bituminous-coated chips; the bottom layer, composed of 3½ in. (loose) of 1¼ to 2½-in. slag, was then rolled; the second course, consisting of 2½ in. (loose) of ½-in. to 1¼-in. slag, was next thoroughly compacted; the third course, composed of ½ in. of ¼ to ½-in. slag chips, was likewise thoroughly rolled; the pavement was finished by rolling in a top dressing of uncoated slag screenings. It may be of interest to note that a section on Madeira Drive, at this famous shore resort, was constructed in this manner in 1903. Its condition in 1910 was excellent, and an adjoining section of water-bound macadam, which was treated annually with an application of tar, rapidly disintegrated before the close of each season.

As an illustration of the practice in Germany, the work of the Deutsche Quarrite and Bitulithic-Pflaster-Gesellschaft of Berlin may be cited. In 1910 this company built a bituminous concrete pavement in Lankwitz, Germany, in accordance with the following method: On a 5-in. macadam cinder-filled foundation, the first course, consisting of 4 in. (loose) of 1½-in. stone, was spread and rolled; a thin coat of tarred chips was next spread and rolled; the second course consisted of 1 in. of ¾-in. stone, well rolled; the final course was ½ in. of tarred chips, also thoroughly rolled; the pavement was finished with a flush coat of tar, on top of which was spread uncoated screenings.

Mr. Blanchard.

That the layer method of constructing bituminous concrete pavements in England is not of recent date is shown by the following description published in 1898:

"The hot stone, when ready for mixing, is screened into material of three sizes, 1 to 2-in. for the body, ½ to 1-in. for the intermediate coat, and ¼ to ½-in. for the top dressing. The coarsest material is used in a layer from 3 to 4 in. thick, the intermediate size forms a coat of about ¾ in., and the top dressing is used in the thinnest layer possible, with a view to filling all interstices. Afterward a dressing of ¼-in. and smaller granite screenings is scattered broadcast, and the traffic is at once allowed on the road to work this top dressing into the tarred material. Each of the layers is rolled separately with a 10-ton roller."

In conclusion, the opinion is expressed that, provided a bituminous cement is mixed with the stone for the second and third courses, the use of the principles of construction embodied in this interesting paper will produce a pavement which, under conditions for which it is suitable, will be economical in construction and maintenance.

Mr. J. L. Meem.

J. L. MEEM, ASSOC. M. AM. SOC. C. E. (by letter).—In reply to Mr. Frink the writer wishes to state that his method deals primarily with the manner of placing the different grades of stone, assuming that the sub-grade is in proper condition to receive them. He agrees with Mr. Frink that the preparation of the sub-grade, drainage, etc., are prime factors in the life of the road, but these are conditions to be controlled locally by the engineer in charge.

The writer is pleased to note that Mr. Sweetser has tried and obtained good results by this method of separating the dust from the screenings and applying them in separate layers. Mr. Sweetser says, in reference to building a road which will be affected less by automobile traffic: "Why not construct at once, so that it will not be affected at all?" The writer is not a champion of "water-bound macadam" roads, and realizes that it is the least durable form of stone pavement; but, as it is the generally accepted form of highway improvement, as adopted by the various counties, due to low first cost, he believes that any improvement in this method of construction without additional cost will be welcomed.

The writer feels that Mr. Crosby's failure to obtain good results by this method of proportioning the rock was due to the fact that he allowed a portion of the chips (¼ to ½ in. in size) to be taken for other purposes, thereby failing to fill the voids in the No. 2 stone. A section of roadway built with the foregoing proportions of rock shows practically a solid mass.

Mr. J. C. Meem's suggestion, that a frequent sprinkling or application of oil as a dust preventive would prolong the life of the road, is a

Mr. J. L. Meem.

good one. This would be of great benefit, especially during long dry periods, but having done this, we have departed from the true water-bound macadam, and Mr. Sweetser's query "Why not construct a permanent road?" presents itself.

Mr. Blanchard's interesting discussion applies to bituminous macadam rather than to water-bound macadam, as set forth in the paper.

The writer is pleased to have obtained the views of others on this simple little change in an accepted specification, and hopes it will at least have the effect of causing others to try the change.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1262

EXPERIMENTAL DETERMINATION OF LOSS OF HEAD DUE TO SUDDEN ENLARGEMENT IN CIRCULAR PIPES.

By W. H. Archer, Esq.

This paper presents a summary of the experimental data and computations therefrom obtained in a series of tests to determine the loss of hydraulic head due to sudden enlargement in cross-section of circular pipes. The question arose in connection with the theory of the jet pump, and its investigation as a thesis led to some interesting conclusions which are herein set forth. Observations were made during December, 1911, and January to March, inclusive, 1912, at the Hydraulic Laboratory, University of California.

Theory.

Theoretically, the determination of loss of head in liquids due to sudden enlargement in cross-section is simple and apparently exact. It is perhaps for this reason that little experimental work has been done to determine the actual losses. For convenience, this theory is here given, and is as follows:

Let v_1 = velocity, in feet per second, at outlet of smaller pipe;
a_1 = cross-sectional area, in square feet, at outlet of smaller pipe;
p_1 = pressure, in pounds per square inch, at outlet of smaller pipe;

γ = density of water, in pounds per cubic foot.

Then $\frac{p}{\gamma}$ = pressure head, in feet of water;

v_2 = velocity, in feet per second, in large pipe, at section where p_2 is a maximum;

p_2 = maximum pressure, in pounds per square inch, in larger pipe;

a_2 = cross-sectional area, in square feet, where p_2 is a maximum;

$l.h.$ = theoretical lost head, in feet of water, due to sudden enlargement;

Q = quantity of water flowing, in cubic feet per second.

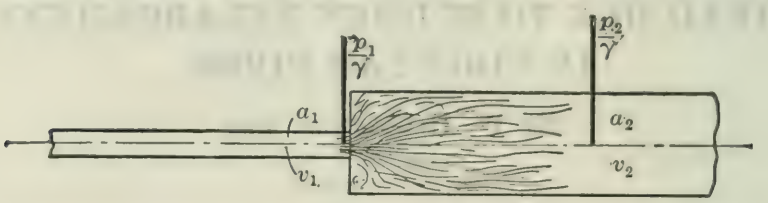


FIG. 1.

The energy relation is:

$$\frac{v_1^2}{2g} + \frac{p_1}{\gamma} = \frac{v_2^2}{2g} + \frac{p_2}{\gamma} + l.h. \quad \ldots\ldots(1)$$

or

$$\frac{v_1^2}{2g} - \frac{v_2^2}{2g} = l.h. + \frac{p_2}{\gamma} - \frac{p_1}{\gamma} \quad \ldots\ldots(2)$$

or

$$\frac{p_2 - p_1}{\gamma} = \frac{v_1^2 - v_2^2}{2g} - l.h \quad \ldots\ldots(3)$$

The momentum relation is as follows:

Force = change in momentum, or $(p_2 - p_1)\, a_2 = \frac{Q}{g}(v_1 - v_2) \ldots\ldots(4)$

By combining Equations (3) and (4) and making continuity substitutions, it is seen that

$$l.h. = \frac{(v_1 - v_2)^2}{2g} \quad \ldots\ldots(5)$$

or, in another form,

$$l.h. = \frac{v_1^2}{2g}\left(1 - \frac{a_1}{a_2}\right)^2 \quad \ldots\ldots(6)$$

The equation for the value of the actual experimental loss will probably be of the form,

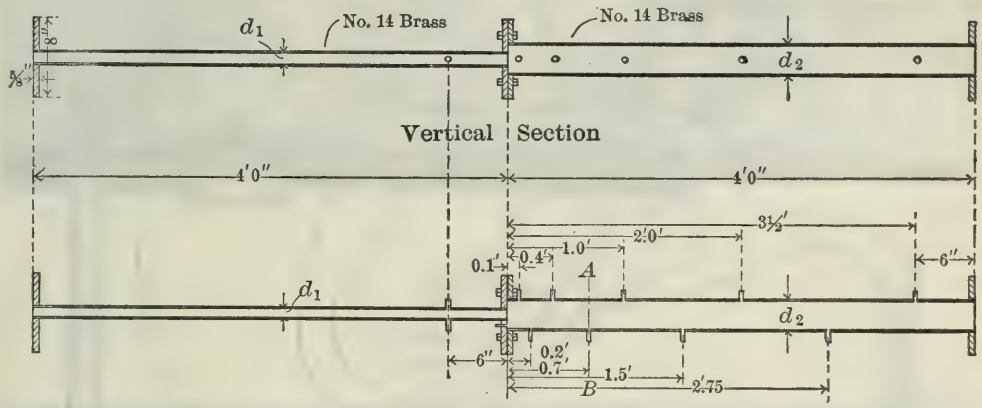
$$L.H. = C\,\frac{v_1^n}{2g}\left(I - \frac{a_1}{a_2}\right)^n \quad \ldots\ldots(7)$$

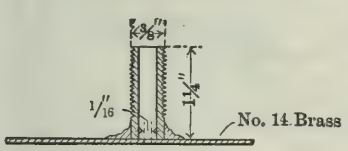
C and n being empirical coefficients, to determine the value of which under various conditions was the purpose of the work described.

Equipment Used.

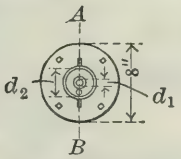
Pipes.—Four sizes of commercial, smooth, brass pipes were used, ranging from 1 to 3 in. in diameter, all being $\frac{1}{16}$ in. thick and 4 ft. long. The ends were machined square and even, and burrs were removed with emery paper.

Flanges.—Connections were made with cast-iron flanges, 8 in. in diameter and $\frac{5}{8}$ in. thick, machined to make a water-tight friction fit over the ends of the brass pipe, on which they were driven until just flush. Flanges were bolted together with gaskets between.





Section of Pressure Tap



Cross-Section

DETAILS OF ENLARGEMENT AND PRESSURE TAPS

FIG. 2.

Pressure Taps.—Taps were inserted in the sides of the pipe, by which internal pressures were obtained. Taps No. 0 in the small pipe were 6 in. from the plane of enlargement and on opposite sides of the pipe. Taps in the larger pipe were 0.1, 0.2, 0.4, 0.7, 1.0, 1.5, 2.0, 2.75, and 3.5 ft. from the plane of enlargement, and were alternated, that is, No. 2 on one side, No. 3 on the opposite side, etc. Taps were made by drilling $\frac{1}{16}$-in. holes (care being taken to drill into wood so as to leave no burrs inside) and then soldering over these holes pieces of $\frac{1}{8}$-in. brass tube $1\frac{1}{2}$ in. long (Fig. 2). Rubber tubing connected these to the manifold and pressure columns.

Manifold.—The manifold consisted of twelve ⅛-in. air-cocks screwed into a piece of ½-in. brass tubing 10 in. long. Five cocks were placed on each side for the connections from the pressure taps in the pipe, and one was put on each end for the pressure column connections.

Pressure Columns or U*-Tubes.*—Connections were made for using either of two U-tubes, depending on the magnitude of pressure difference. One tube, with arms about 3 ft. long, partly filled with mercury, was used for large differences in pressure. The other, having a short arm, 3½ ft., and a long arm, 5 ft. in length, was partly filled with carbon tetra chloride colored red with an aniline dye. Both U-tubes were made entirely of ¼-in. glass tubing, it being found that the carbon tetra chloride acted on the rubber, causing it to swell until the passage was closed.

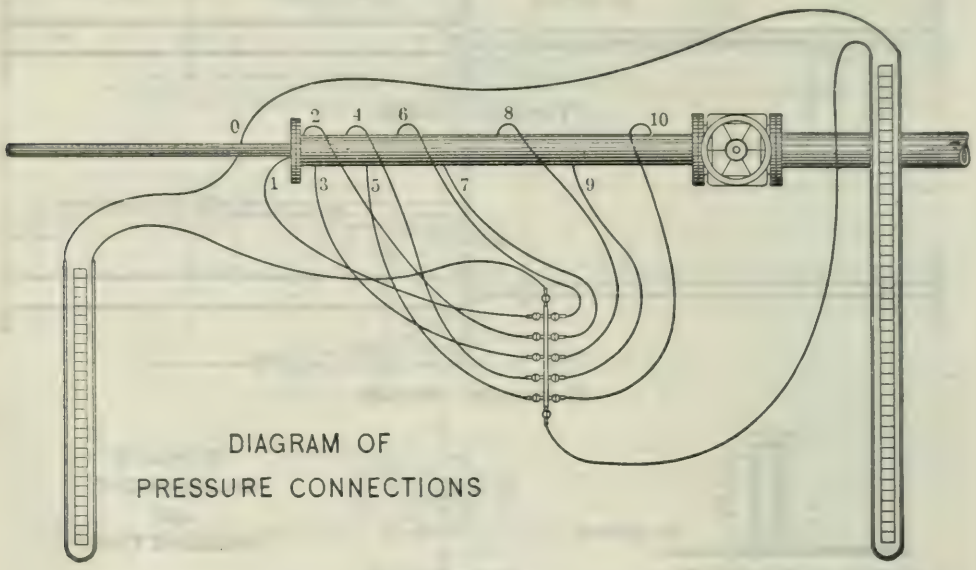


FIG. 3.

In order to obtain a good meniscus on the surface of carbon tetra chloride it is necessary that the glass be absolutely clean. This cleaning was done with a saturated solution of bichromate of potash one part, sulphuric acid three parts and then diluting with six parts of water. It was found necessary to flush all parts of the U-tube with running water to remove the cleaning solution thoroughly, otherwise a bothersome scum was formed on the surface of the carbon tetra chloride.

Bleeder cocks were placed at the upper ends of each arm of the U-tubes, to allow for the expulsion of air from the tubes and connec-

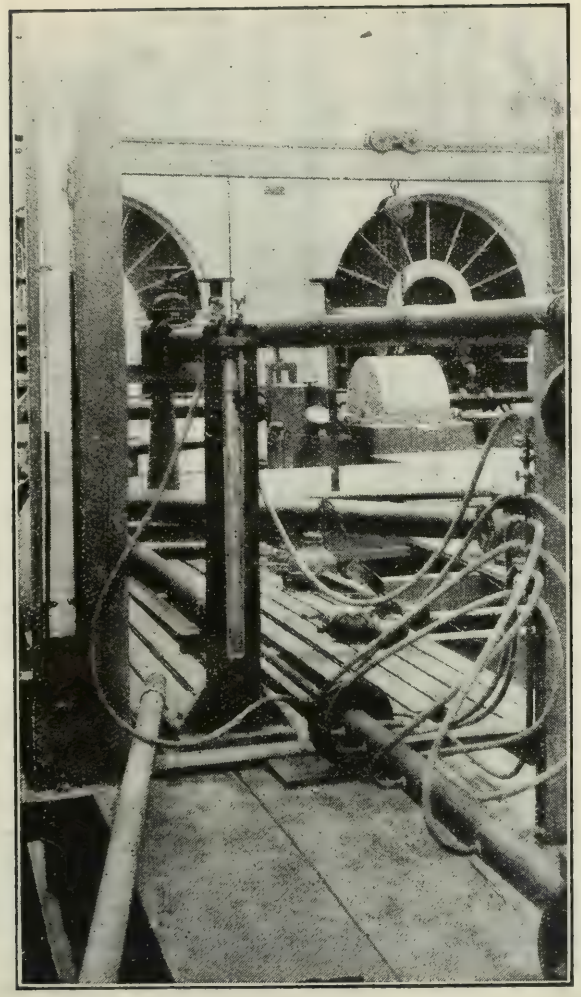


FIG. 4.—VIEW OF APPARATUS FROM OUTLET END.

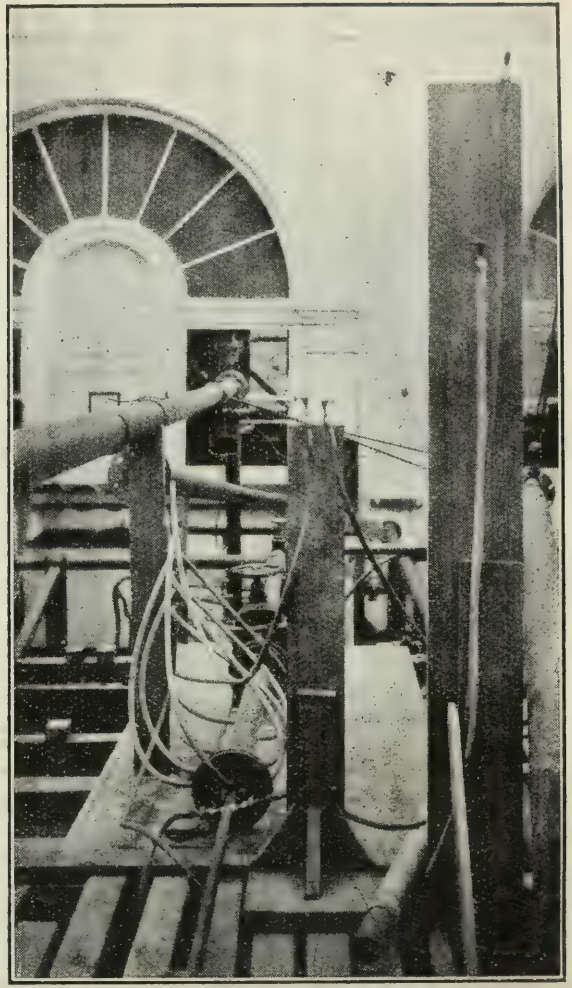


FIG. 5.—VIEW OF APPARATUS FROM INLET END.

tions. The scales between the arms of the U-tubes were graduated in inches and tenths.

Water Supply.—Water was obtained for the experiments through a horizontal 4-in. pipe about 75 ft. long leading from the laboratory stand-pipe, which is 4 ft. in diameter and 36 ft. high (Fig. 7). The height of water in the stand-pipe was maintained with a motor-driven Quimby screw pump (Fig. 6), the two cylinders working in series, and drawing water from the main storage reservoir, which is a concrete tank, 18 ft. square and 10 ft. deep, located beneath the floor. Water is kept at varying heights by overflows from the stand-pipe through any required gate in the side thereof, such water returning to the storage reservoir.

Quantity Measurements.—The water was taken from the throttle valve, at the outlet end of the large brass pipe, into a $2\frac{1}{2}$-in. pipe which led to a Y. One arm of the Y discharged into a galvanized-iron tank on platform scales, and the other discharged either into the main reservoir or into a calibrated measuring tank, depending on which way the hinged piece of pipe at the end was swung.

The tank on the scales, with a capacity of about 25 cu. ft., was used for measuring small quantities of water. It was emptied into the main reservoir by a quick-opening valve.

Larger quantities were measured in the calibrated concrete tank, 7 ft. square and 10 ft. deep, using a hook-gauge. A centimeter scale on the hook-gauge was used for a few readings, and an inch scale for the remainder. This tank was also emptied into the main storage reservoir by a 4-in. motor-driven centrifugal pump.

Preliminary Observed Data.—Due to the fact that the iron flanges were driven on the brass pipes, the end diameter was slightly decreased in some cases. The central diameter is taken as the diameter before the flanges were put in place. All measurements were made with micrometer calipers.

No. of pipe.	Average Diameter, in Inches.			Average Areas, in Square Feet.		
	Inlet.	Center.	Outlet.	Inlet.	Center.	Outlet.
1.....	.....	0.992	0.982	.....	0.00545	0.00526
2.....	1.735	1.745	1.730	0.01641	0.01661	0.01631
3.....	2.488	2.495	2.486	0.03378	0.03395	0.03368
4.....	2.983	2.994	2.993	0.04855	0.04885	0.04885

Letting a_1 represent the outlet area of the small pipe and a_2 the central area of the large pipe, the following values were taken:

No. 1 $a_1 = 0.00526$; $a_2 = 0.00545$
No. 2 $a_1 = 0.01631$; $a_2 = 0.01661$
No. 3 $a_1 = 0.03368$; $a_2 = 0.03395$
No. 4 $a_1 = 0.04885$; $a_2 = 0.04885$

Specific Gravity Determination of Carbon Tetra Chloride.—The specific gravity of the carbon tetra chloride was found by taking the ratio of the length of column of water in one arm of a U-tube to the length of column of carbon tetra chloride in the other arm which balanced it.

The average value of four determinations for all the experiments was 1.601, with the exception of the $1\frac{3}{4}$ to $2\frac{1}{2}$-in. combination of pipes, the specific gravity for which (average of three determinations) was 1.570.

These values were used in computing the pressure change, $\frac{p_2 - p_1}{\gamma}$, as follows:

Let S = specific gravity of liquid in U-tube;
Let U_1 = scale reading in arm of U-tube, where pressure = p_1;
Let U_2 = scale reading in arm of U-tube connected to tap where pressure = p_2.

Then:

If U_1 is greater than U_2, the column of liquid of length $U_1 - U_2$ is balanced by a column of water of length $U_1 - U_2$ and by $\frac{p_2 - p_1}{\gamma}$.

Therefore:

$$\frac{p_2 - p_1}{\gamma} = (S - 1)(U_1 - U_2).$$

In all the determinations, the zero positions, U_0, of the liquid in the columns were carefully noted where $p_1 = p_2$, or when no water was flowing. Then, in most observations, U_1 only was read; and, in nearly all computations, $U_1 - U_2$ was taken as being $2(U_1 - U_0)$. Under the circumstances, this was considered more accurate because there was a perfect meniscus in the arm, U_1, while the surface in the arm, U_2, was broken and irregular. Some readings of U_2 were taken occasionally as a check.

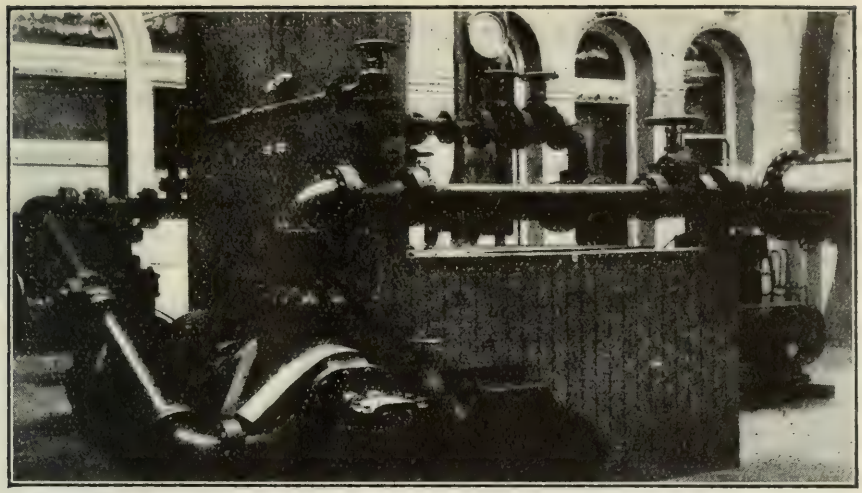


FIG. 6.—QUIMBY SCREW PUMP.
HYDRAULIC LABORATORY, UNIVERSITY OF CALIFORNIA.

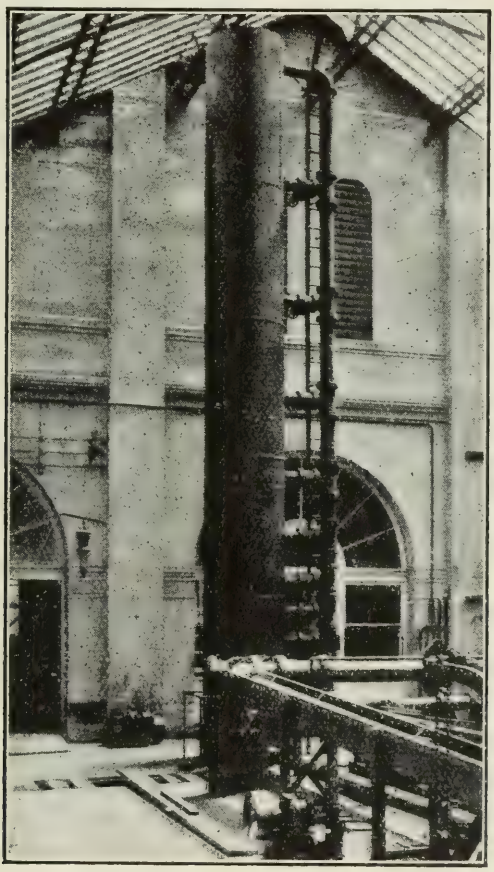


FIG. 7.—STAND-PIPE, HYDRAULIC LABORATORY, UNIVERSITY OF CALIFORNIA.

Tables 1 to 5, inclusive, show the values of $\frac{p_2 - p_1}{\gamma}$ computed from readings of U_1 and U_0, and the relation between velocity and $\frac{p_2 - p_1}{\gamma}$ at different points along the length of the large pipe is shown graphically on Figs. 8 to 12, inclusive.

Miscellaneous Data and Constants.—

Specific gravity of mercury..... 13.58
Density of water..... 62.3 lb. per cu. ft.
Measuring tank constants:
1 cm. gauge height..... 1.523 cu. ft.
1 in. gauge height..... 3.868 " "
Head in stand-pipe above apparatus:
No. 7, Head ..... 19 ft.
No. 8, " ..... 25 "
No. 9, " ..... 31 "

Method and Order of Taking Observations.

The Quimby screw pump was started, and, when the water was overflowing at the desired gate in the stand-pipe, the maximum range of U_1, the scale reading in the arm of the U-tube connected to the small pipe, was determined for maximum velocity and for some pressure tap in the large pipe where p_2 was near a maximum, say, Tap No. 6. Then, as it was intended to make observations for about ten different velocities for each combination, this range of U_1 was divided into ten steps, and the velocity for each determination was to be regulated by the gate-valve at the outlet end of the large pipe, so that for each successive trial, and for pressure Tap No. 6, the value of U_1 would increase by about one-tenth.

When the velocity had been thus regulated, the initial position of the surface of the water in the measuring tank was observed with a hook-gauge.

The hands of a watch were then set at some even hour, and the instant when the second hand was at 60 the swing pipe was turned so as to divert the water from flowing into the main reservoir and into the measuring tank.

While the water was flowing into the measuring tank at a constant velocity, readings of U_1 and U_2 were taken, when the various cocks in

TABLE 1.

$d_1 = 2.486$ in.; $d_2 = 2.994$ in. Area ratio = 1.451.

Number of Trial.		1	2	3	4	5	6	7	8	9
Velocity, v_1		3.885	7.580	11.060	13.310	5.970	10.000	13.020	14.590	16.580
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various pressure taps.....	No. 1	.....	.....	.....	.....	.....	.....	.....	.....	.....
	No. 2	+0.007	+0.007	+0.005	+0.015	+0.010	+0.012	+0.020	+0.025	+0.051
	No. 3	0.045	0.179	0.394	0.591	0.111	0.326	0.562	0.733	0.915
	No. 4	0.060	0.266	0.577	0.861	0.162	0.477	0.822	1.054	1.370
	No. 5	0.070	0.282	0.631	0.942	0.181	0.522	0.892	1.152	1.490
	No. 6	0.072	0.301	0.648	0.967	0.187	0.535	0.916	1.174	1.520
	No. 7	0.070	0.296	0.633	0.948	0.183	0.520	0.842	1.153	1.486
	No. 8	0.066	0.278	0.590	0.886	0.173	0.490	.....	1 075	1.380
	No. 9	0.058	0.248	0.524	0.787	0.153	0.437	.....	0.956	1.240
	No. 10	0.051	0.220	0.475	0.726	0.133	0.395	.....	0.872	1.136

TABLE 2.

$d_1 = 1.730$ in.; $d_2 = 2.495$ in. Area ratio = 2.077.

Number of Trial.		1	2	3	4	5	6	7	8	9	10
Velocity, v_1		3.820	8.000	10.340	12.530	14.350	16.000	17.380	18.610	20.030	21.050
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various taps.	No. 1	−0.022	−0.044	−0.059	−0.077	−0.084	−0.085	−0.099	−0.100	−0.106	−0.104
	No. 2	−0.021	−0.052	−0.074	−0.101	−0.123	−0.133	−0.151	−0.158	−0.162	−0.162
	No. 3	+0.007	+0.095	+0.168	+0.265	+0.367	+0.476	+0.587	+0.675	+0.787	+0.888
	No. 4	0.034	0.289	0.496	0.745	1.010	1.262	1.532	1.775	2.050	2.280
	No. 5	0.046	0.381	0.638	0.965	1.300	1.622	1.950	2.235	2.590	2.840
	No. 6	0.051	0.415	0.700	1.040	1.395	1.738	2.095	2.375	2.755	3.030
	No. 7	0.051	0.413	0.696	1.032	1.391	1.730	2.080	2.365	2.745	3.020
	No. 8	0.049	0.402	0.680	1.010	1.354	1.690	2.035	2.329	2.690	2.980
	No. 9	0.046	0.365	0.621	0.923	1.235	1.538	1.853	2.120	2.470	2.740
	No. 10	0.043	0.363	0.618	0.923	1.240	1.548	1.860	2.125	2.480	2.740

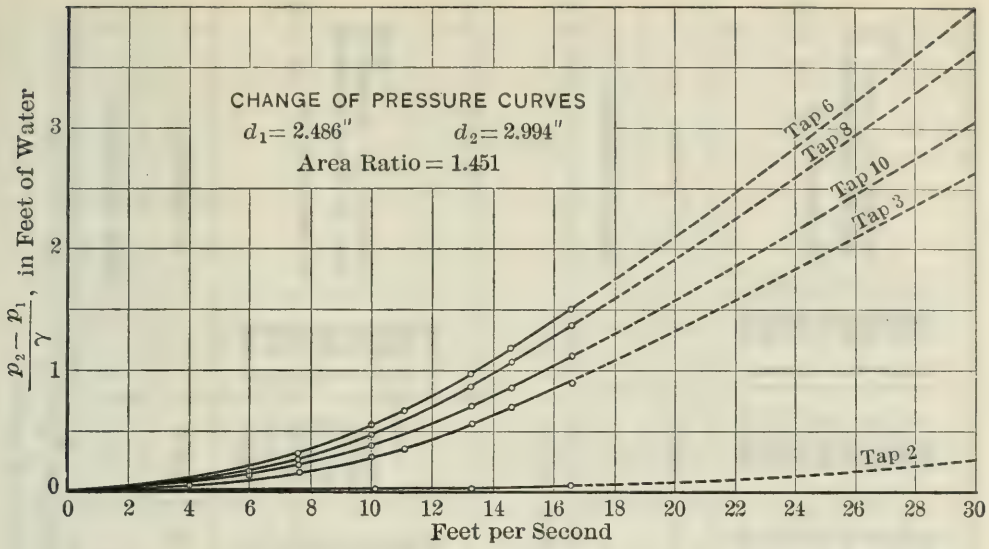

CHANGE OF PRESSURE CURVES
$d_1 = 2.486''$ $d_2 = 2.994''$
Area Ratio = 1.451
Tap 6
Tap 8
Tap 10
Tap 3
Tap 2
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second
0 1 2 3
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30

FIG. 8.

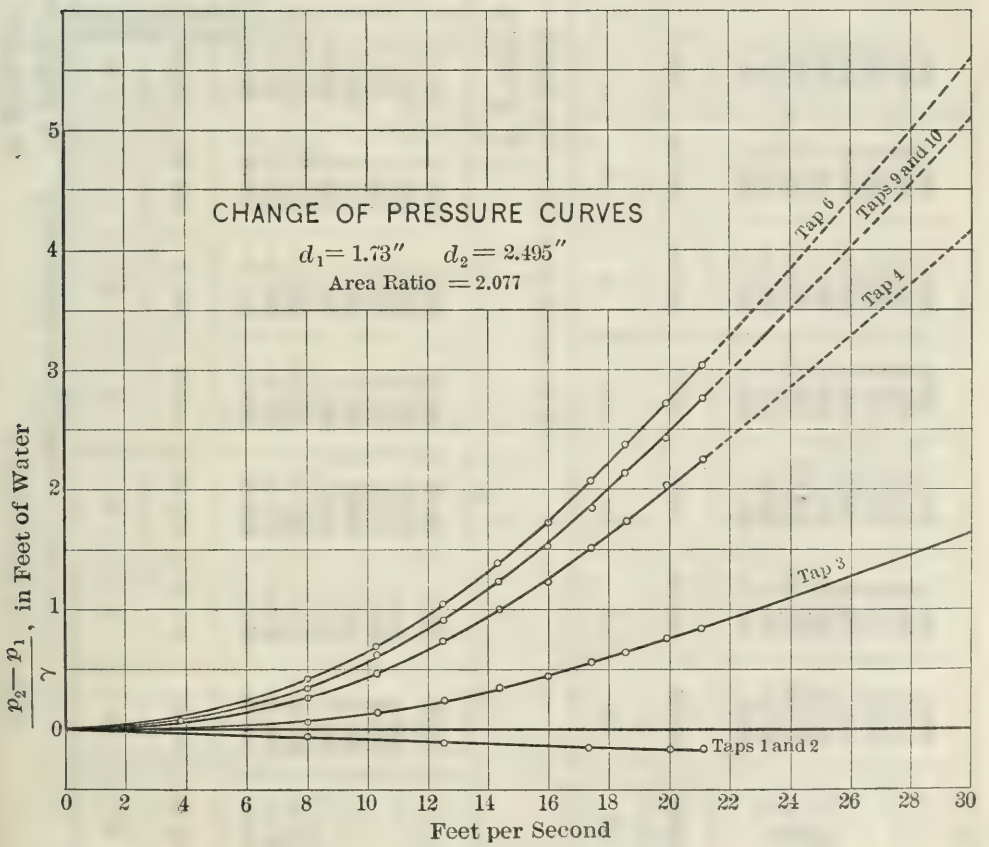

CHANGE OF PRESSURE CURVES
$d_1 = 1.73''$ $d_2 = 2.495''$
Area Ratio = 2.077
Tap 6
Taps 9 and 10
Tap 4
Tap 3
Taps 1 and 2
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second
0 1 2 3 4 5
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30

FIG. 9.

TABLE 3.

$d_1 = 1.730$ in.; $d_2 = 2.994$ in. Area ratio = 2.995.

Number of Trial.		1	2	3	4	5	6	7	8	9	10	11
Velocity, v_1		5.410	7.520	10.250	12.960	15.280	17.120	18.770	20.770	21.120	21.080	23.920
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various taps.	No. 1	—0.027	—0.039	—0.052	—0.060	—0.069	—0.079	—0.081	—0.078	—0.075	—0.075	—0.070
	No. 2	—0.031	—0.044	—0.063	—0.082	—0.092	—0.108	—0.115	—0.123	—0.125	—0.125	—0.130
	No. 3	—0.026	—0.038	—0.045	—0.051	—0.049	—0.056	—0.051	—0.048	—0.030	—0.040	—0.030
	No. 4	+0.045	+0.123	+0.228	+0.415	+0.610	+0.776	+0.960	+1.200	+1.250	+1.230	+1.600
	No. 5	0.128	0.273	0.535	0.880	1.275	1.590	1.960	2.400	2.530	2.480	.....
	No. 6	0.154	0.336	0.645	1.060	1.510	1.890	2.300	2.840	2.980	2.920	.....
	No. 7	0.163	0.351	0.665	1.090	1.550	1.940	2.410	2.910	3.050	2.990	.....
	No. 8	.....	0.340	0.655	1.070	1.540	1.920	2.390	2.860	3.010	2.960	.....
	No. 9	.....	0.337	0.640	1.100	1.500	1.870	2.280	2.810	.....	.....	.....
	No.10	.....	0.325	0.614	1.060	1.450	1.800	2.220	2.720	2.880	2.770	.....

TABLE 4.

$d_1 = 0.982$ in.; $d_2 = 2.495$ in. Area ratio = 6.46.

Number of Trial.		1	2	3	4	5	6	7	8	9	10	11
Velocity, v_1		7.020	13.710	19.560	21.380	23.900	24.580	25.120	25.390	25.300	25.820	28.970
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various taps.	No. 1	—0.066	—0.120	—0.153	—0.177	—0.180	—0.175	—0.189	—0.200	—0.134	—0.027	.....
	No. 2	—0.071	—0.135	—0.183	—0.234	—0.252	—0.256	—0.269	—0.280	—0.249	—0.289	—0.156
	No. 3	—0.071	—0.134	—0.179	—0.237	—0.238	—0.241	—0.250	—0.256	—0.230	—0.251	—0.270
	No. 4	+0.014	+0.025	+0.110	+0.386	+0.558	+0.602	+0.615	+0.627	+0.630	+0.633	+0.860
	No. 5	0.108	0.343	0.656	1.430	1.875	1.940	2.020	2.065	2.090	2.100	2.760
	No. 6	0.127	0.406	0.748	1.630	2.110	2.200	2.310	2.345	2.370	2.380	.....
	No. 7	0.127	0.405	0.755	1.640	2.120	2.210	2.315	2.310	2.375	2.390	.....
	No. 8	0.126	0.402	0.753	1.630	2.100	2.200	2.310	2.220	2.370	2.380	.....
	No. 9	0.126	0.396	0.740	1.600	2.080	2.170	2.270	2.110	2.335	2.375	.....
	No. 10	0.126	0.395	0.736	1.595	2.050	2.155	2.265	2.000	2.310	2.380	.....

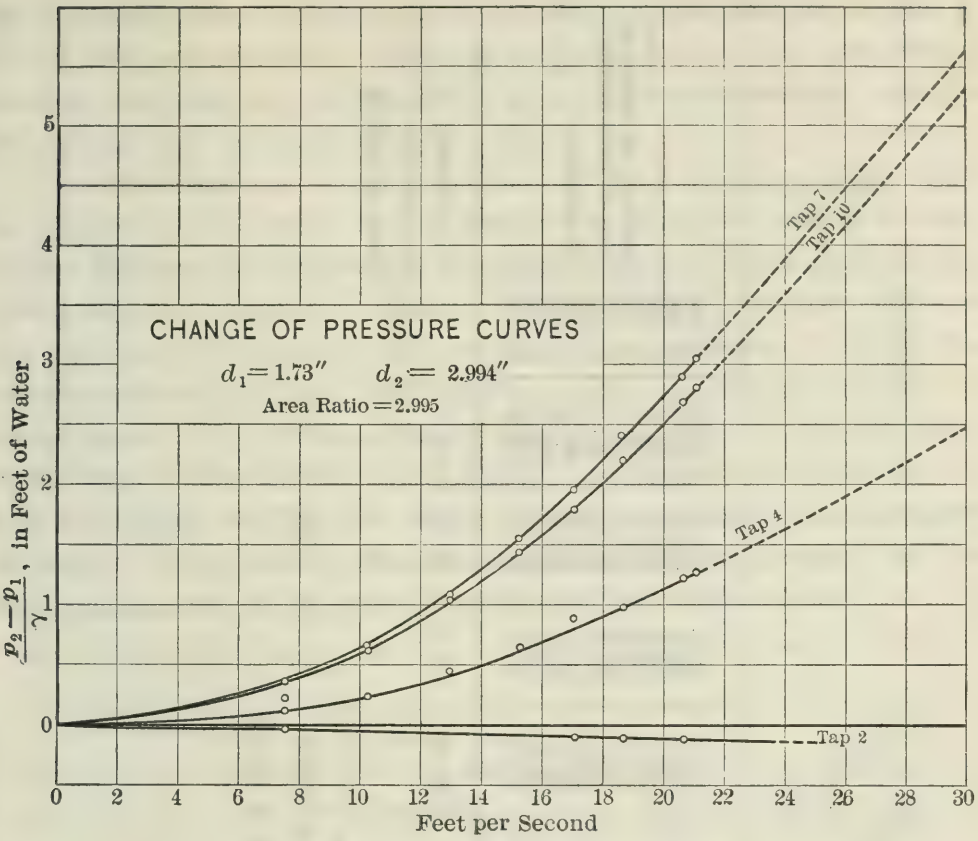

CHANGE OF PRESSURE CURVES
$d_1 = 1.73''$ $d_2 = 2.994''$
Area Ratio = 2.995
Tap 7
Tap 10
Tap 4
Tap 2
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second

FIG. 10.

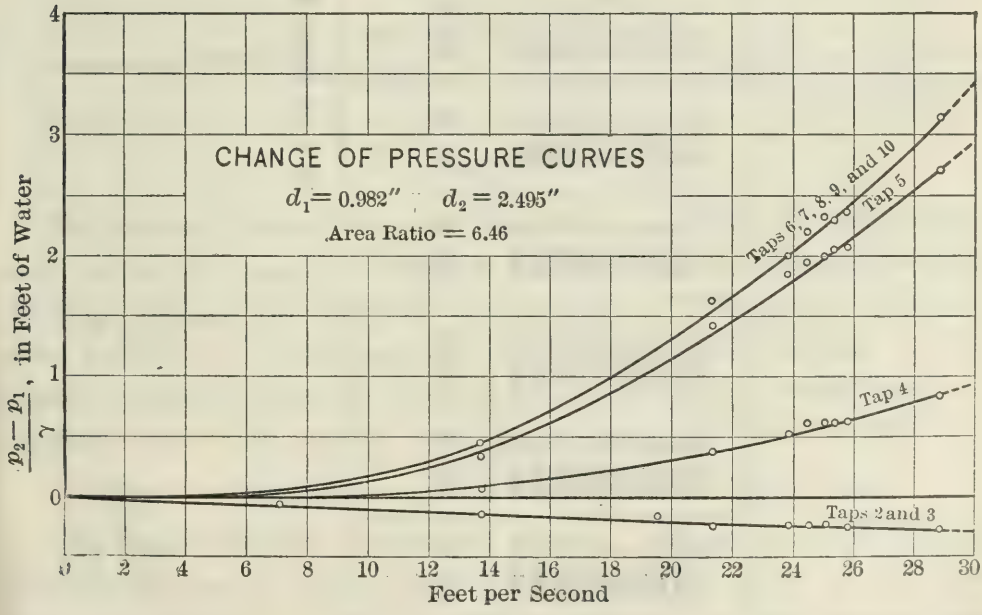

CHANGE OF PRESSURE CURVES
$d_1 = 0.982''$ $d_2 = 2.495''$
Area Ratio = 6.46
Taps 6, 7, 8, 9, and 10
Tap 5
Tap 4
Taps 2 and 3
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second

FIG. 11.

TABLE 5.

$d_1 = 0.982$ in.; $d_2 = 2.994$ in. Area ratio = 9.22.

Number of Trial.		1	2	3	4	5	6	7	8	9	10	11
Velocity, v_1		6.919	10.370	13.060	14.730	16.410	20.650	25.030	20.540	24.850	27.230	31.580
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various taps.	No. 1	−0.070	−0.119	−0.147	−0.170	−0.173	−0.196	−0.213	−0.193	−0.211	−0.196	−0.193
	No. 2	−0.075	−0.119	−0.161	−0.189	−0.193	−0.228	−0.258	−0.223	−0.251	−0.266	−0.253
	No. 3	−0.075	−0.119	−0.159	−0.183	−0.193	−0.233	−0.253	−0.218	−0.261	−0.261	−0.238
	No. 4	−0.065	−0.101	−0.116	−0.125	−0.132	−0.097	−0.077	−0.107	−0.070	−0.051	+0.018
	No. 5	+0.028	+0.114	+0.215	+0.294	+0.425	+0.725	+1.140	+0.735	+1.130	+1.380	2.005
	No. 6	0.075	0.207	0.360	0.481	0.655	1.092	1.690	1.082	1.680	2.030	2.830
	No. 7	0.081	0.218	0.380	0.520	0.678	1.130	1.740	1.110	1.730	2.110	2.920
	No. 8	0.080	0.219	0.380	0.514	0.680	1.130	1.750	1.115	1.740	2.120	2.920
	No. 9	0.078	0.216	0.376	0.512	0.671	1.126	1.740	1.113	1.728	2.100	2.910
	No. 10	0.076	0.215	0.378	0.512	0.670	1.118	1.672	.....	1.710	2.080	2.900

the manifold were turned so as to connect pressure taps No. 1, No. 2, No. 3, etc., successively. Then, at the end of an even number of minutes, the water was diverted back into the storage reservoir, and the hook-gauge was set approximately right.

The adjustments of velocity for the next trial were then made while the surface of the water in the measuring tank was coming to rest. Before this was done, however, the controlling gate-valve was entirely closed and the reading U_0 taken. If the liquid in the two columns did not come to the same level, it showed the presence of air in the pressure connections, and doubt was cast on the accuracy of the previous observations. The air was then expelled by opening the bleeder cocks at the tops of the U-tube. The velocity was regulated as before, the final tank gauge reading was taken, and the next observation continued as before. Only two or three times during the experiment did the two column arms fail to balance because of the presence of air.

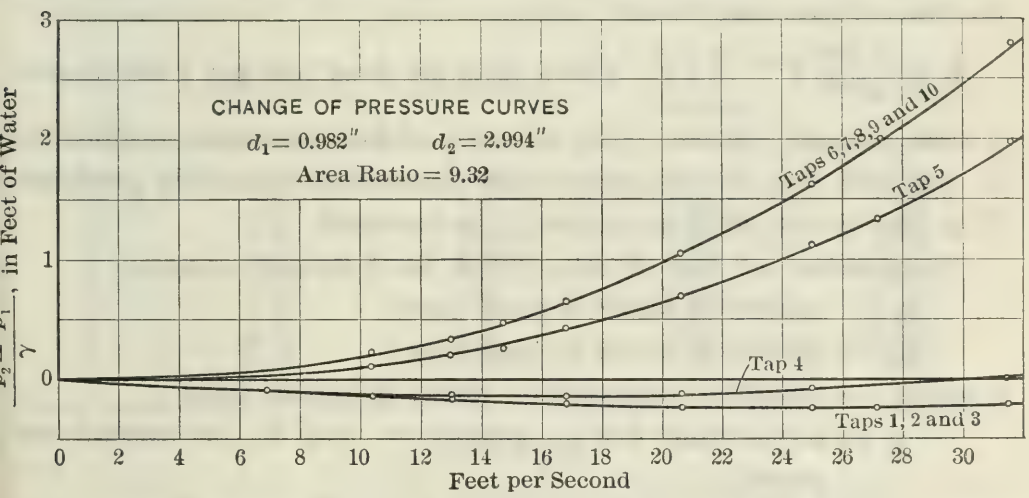

CHANGE OF PRESSURE CURVES
$d_1 = 0.982''$ $d_2 = 2.994''$
Area Ratio = 9.32
Taps 6, 7, 8, 9 and 10
Tap 5
Tap 4
Taps 1, 2 and 3
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30
0 1 2 3

FIG. 12.

The mercury column was used but little, as nearly all the readings were within the range allowed by the long arm of the carbon tetra chloride column, and because the readings of the mercury column were subject to inaccuracy as the rubber tube leading to the U-tube came from the top of the manifold so that it would catch any air that might leak into the manifold or tubes connected thereto.

Friction Corrections and Computation of Lost Head.

The experimentally determined value of change in pressure head includes the loss of head due to friction in 6 in. of the smaller pipe and in the length, X, of the larger pipe, where X = the distance from the plane of enlargement to the point of maximum pressure, p_2.

To correct for this, the friction slope for each pipe was computed for several velocities, and lines were plotted on logarithmic paper showing the value of the slope for any velocity in any of the pipes used.

The formula adopted was :*

$$S^{0.54} = \frac{v}{1.32\ C\ r^{0.63}};$$

S = fall, in feet, in friction gradient per foot of length;

v = velocity, in feet per second ;

C = constant, depending on surface of pipe ;

r = hydraulic radius = $\left(\frac{d}{4}\right)$ feet for pipes.

The value of C given by Williams and Hazen for smooth brass pipes is from 135 to 145; 140 was taken.

This formula is based on experimental work† done by A. V. Saph, M. Am. Soc. C. E., and E. W. Schoder, Assoc. M. Am. Soc. C. E. For brass pipes they found:

$H = \frac{0.296}{d^{1.25}}\ V^{1.75} = 7\%$, where H is the head lost per 1 000 ft. and d is the diameter, in feet. The accuracy of the corrections is indicated by the fact that the experimentally determined hydraulic gradients (Fig. 14) become horizontal when thus corrected.

Computation of Lost Head.—Using the following notation:

v_1 = velocity of water in small pipe;

v_2 = velocity of water in large pipe;

a_1 and a_2 = respective cross-section areas, in square feet;

g = acceleration due to gravity = 32.2 ft. per second per second;

h = total friction head correction;

$\frac{p_2 - p_1}{\gamma}$ = measured change in pressure head;

$L.H.$ = actual experimental lost head due to sudden enlargement.

$$L.H. = \frac{{v_1}^2 - {v_2}^2}{2g} - \frac{p_2 - p_1}{\gamma} - h;$$

but $$\frac{{v_1}^2 - {v_2}^2}{2g} = \frac{{v_1}^2}{2g}\left(1 - \frac{(a_1)^2}{(a_2)}\right)$$

Therefore, $$L.H. = \frac{{v_1}^2}{2g}\left(1 - \frac{(a_1)^2}{(a_2)}\right) - \frac{(p_2 - p_1)}{\gamma} - h.$$

* Taken from Williams and Hazen's "Hydraulic Tables."

† *Transactions*. Am. Soc. C. E., Vol. LI, p. 253.

Using this formula, the results are shown in Tables 6 to 10, inclusive.

TABLE 6.

$d_1 = 2.486$ in.; $d_2 = 2.944$ in. Area ratio = 1.451.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1.....	3.885	2.680	0.072	0.038
2.....	7.580	5.220	0.301	0.098
3.....	11.060	7.620	0.648	0.212
4.....	13.310	9.180	0.967	0.280
5.....	5.970	4.120	0.187	0.060
6.....	10.000	6.900	0.535	0.163
7.....	13.020	8.980	0.916	0.286
8.....	14.590	10.060	1.174	0.330
9.....	16.580	11.440	1.520	0.429

TABLE 7.

$d_1 = 1.730$ in.; $d_2 = 2.495$ in. Area ratio = 2.077.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1.....	3.82	1.837	0.0513	0.108
2.....	8.00	3.842	0.4150	0.259
3.....	10.34	4.970	0.700	0.434
4.....	12.53	6.020	1.04	0.624
5.....	14.35	6.900	1.395	0.787
6.....	16.00	7.690	1.738	0.977
7.....	17.38	8.36	2.095	1.120
8.....	18.61	8.94	2.375	1.319
9.....	20.03	9.63	2.755	1.515
10.....	21.05	10.12	3.03	1.694

TABLE 8.

$d_1 = 1.730$ in.; $d_2 = 2.944$ in. Area ratio = 2.995.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1.....	5.41	1.088	0.163	0.208
2.....	7.52	2.510	0.351	0.385
3.....	10.25	3.424	0.660	0.660
4.....	12.96	4.33	1.090	1.033
5.....	15.28	5.105	1.55	1.411
6.....	17.12	5.72	1.94	1.775
7.....	18.77	6.27	2.41	1.982
8.....	.....	.....	.....	.....
9.....	21.12	7.055	3.05	2.635
10.....	.....	.....	.....	.....
11.....	23.92	7.990	4.19	3.100

TABLE 9.

$d_1 = 0.982$ in.; $d_2 = 2.495$ in. Area ratio = 6.46.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1.....	7.02	1.086	0.127	0.510
2.....	.....	.....	.....	.....
3.....	.....	.....	.....	.....
4.....	21.38	3.31	1.640	4.420
5.....	23.90	3.70	2.120	5.461
6.....	24.58	3.805	2.210	5.816
7.....	25.12	3.890	2.310	6.072
8.....	.....	.....	.....	.....
9.....	.....	.....	.....	.....
10.....	25.82	4.00	2.39	6.488
11.....	28.97	4.48	3.20	8.006

TABLE 10.

$d_1 = 0.982$ in.; $d_2 = 2.994$ in. Area ratio = 9.32.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1.....	6.919	0.744	0.0803	0.555
2.....	10.37	1.17	0.219	1.209
3.....	13.06	1.40	0.380	1.894
4.....	14.73	1.58	0.514	2.384
5.....	16.41	1.77	0.680	2.923
6.....	Near duplicate of other trial.			
7.....	25.03	2.69	1.75	6.721
8.....	Near duplicate of other trials.			
9.....	" " " " "			
10.....	27.23	2.93	2.12	7.854
11.....	31.58	3.40	2.92	10.646

Determination of Formula for Lost Head.

These values of v_1 and lost head, Tables 6 to 10, inclusive, were plotted three times, independently, on logarithmic cross-section paper.

The plotted points determined lines which were not only straight but nearly parallel, which indicated that the lost head varied as a constant power of the velocity. The value of this exponent (n) which, of course, was the tangent of the angle of inclination of the parallel lines with the velocity axis, was 1.919, and was determined by taking an average of the fifteen values obtained by the three trials of plotting of five lines each. In short, $L.H.$ varied as $v_1^{1.919}$.

Also: because, by theory, $l.h. = \frac{v_1^2}{2g}\left(1 - \frac{a_1}{a_2}\right)^2$, assume that

$$L.H. = c\, v_1^n \left(1 - \frac{a_1}{a_2}\right)^n.$$

This relation was proven to be true by plotting on logarithmic cross-section paper the values of lost head as ordinates against the corresponding values of $\left(1 - \frac{a_1}{a_2}\right)$ for a constant velocity. All the points lay very close to a straight line with slope = 1.919.

The production of this line to the axis, where $\left(1 - \frac{a_1}{a_2}\right)$ was equal to unity, and thence to such a value that v_1 was unity, gave the value of the constant, c.

From such construction, $C = 0.01705$. Hence the relationship appears to be

$$L.H. = 0.01705\ v_1^{1.919} \left(1 - \frac{a_1}{a_2}\right)^{1.919}$$

or

$$L.H. = 1.098 \frac{v_1^{1.919}}{2g} \left(1 - \frac{a_1}{a_2}\right)^{1.919}.$$

Relation of Theoretical to Actual Loss of Head.

$$L.H. = \text{Actual loss} = K\ V_1^{1.919}$$
$$l.h. = \text{Theoretical loss} = k\ v_1^2$$
$$\frac{L.H.}{l.h.} = \frac{K\ V_1^{1.919}}{k\ v_1^2} = K'\ v_1^{-0.081}$$

or, if $B = K'\ v_1^{-0.081}$ where B is a variable coefficient,

$$\text{then } L.H. = B \times (l.h.) = B \frac{(v_1 - v_2)^2}{2g}.$$

The value of B, or the ratio of actual loss to theoretical loss at any velocity, v_1, may be shown on logarithmic cross-section paper, for any desired ratio of pipe areas, by drawing lines with slope = — 0.081 at such positions that their intersections with lines of actual lost head for the same area ratios occur at velocities of such values that the theoretical loss for corresponding area ratios is unity. In other words, determine the value of v_1 to make the theoretical loss unity for any desired area ratio, and then the value of the actual loss at that velocity is a point on the line with slope = — 0.081, showing the value of B for that area ratio and for any velocity. Table 11 shows the values of B for different area ratios and velocities.

TABLE 11.—VALUE OF COEFFICIENT, B.

$$\text{True lost head} = B\,\frac{(v^1 - v^2)^2}{2g}.$$

Vel. v_1 / $a_1 \div a_2$	2	4	6	8	10	12	15	20	25	30	40	50	60	70	80
1 : 1¼	1.225	1.162	1.123	1.092	1.075	1.060	1.038	1.017	0.994	0.981	0.959	0.941	0.926	0.915	0.903
1 : 1½	1.155	1.087	1.057	1.025	1.001	0.991	0.976	0.953	0.938	0.923	0.901	0.886	0.871	0.860	0.850
1 : 2	1.090	1.030	0.998	0.975	0.958	0.943	0.928	0.905	0.889	0.876	0.855	0.839	0.826	0.816	0.807
1 : 2½	1.068	1.012	0.977	0.954	0.938	0.924	0.906	0.886	0.870	0.857	0.837	0.821	0.809	0.798	0.789
1 : 3	1.063	1.002	0.970	0.947	0.930	0.917	0.900	0.880	0.863	0.849	0.830	0.816	0.802	0.791	0.783
1 : 4	1.055	0.996	0.965	0.943	0.925	0.911	0.893	0.876	0.858	0.846	0.827	0.811	0.799	0.788	0.780
1 : 5	1.049	0.911	0.960	0.938	0.920	0.906	0.890	0.870	0.852	0.840	0.820	0.805	0.792	0.782	0.774
1 : 10	1.040	0.983	0.952	0.930	0.913	0.899	0.883	0.869	0.847	0.835	0.815	0.800	0.788	0.779	0.769
1 : 20	1.033	0.979	0.947	0.925	0.907	0.893	0.878	0.858	0.841	0.829	0.810	0.794	0.782	0.772	0.764
1 : ∞	1.022	0.968	0.937	0.915	0.898	0.884	0.869	0.848	0.832	0.820	0.800	0.785	0.773	0.763	0.754

Hydraulic Gradients.

Tables 12 to 16, inclusive, show the data from change of pressure curves, Figs. 8 to 12, inclusive, for hydraulic gradients, not corrected for friction, the values of $\frac{p_2 - p_1}{\gamma}$ being in feet of water. Fig. 13 shows a typical set of gradients plotted from Table 12.

TABLE 12.—Hydraulic Gradients.

Combination No. 1: $\frac{a_2}{a_1} = 1.451$.

v_1	Tap Number.									
	1	2	3	4	5	6	7	8	9	10
5	.....	+0.003	+0.80	+0.13	+0.15	+0.15	....	+0.12	....	+0.11
10	.....	+0.011	+0.30	+0.43	+0.51	+0.54	....	+0.44	....	+0.39
15	.....	+0.035	+0.77	+1.11	+1.22	+1.23	....	+1.12	....	+0.93
20 (from curves produced)	.....	+0.100	+1.33	+1.90	+2.10	+2.10	....	+2.00	....	+1.60
25 (from curves produced)	.....	+0.176	+1.94	+2.75	+3.04	+3.00	....	+2.96	....	+2.30

TABLE 13.—Hydraulic Gradients.

Combination No. 2: $\frac{a_2}{a_1} = 2.077$.

v_1	Tap Number.									
	1	2	3	4	5	6	7	8	9	10
5	−0.03	−0.028	+0.02	+0.10	+0.11	+0.16	.....	.....	+0.12	....
10	−0.06	−0.075	+0.13	+0.43	+0.61	+0.65	.....	.....	+0.58	....
15	−0.087	−0.128	+0.40	+1.13	+1.43	+1.53	.....	....	+1.36	....
20	−0.105	−0.1625	+0.79	+2.05	+2.59	+2.75	.....	.....	+2.47	....
30 (from curves produced)	.....	.....	+1.75	+4.20	+5.25	+5.60	.....	.....	+5.10	....

TABLE 14.—Hydraulic Gradients.

Combination No. 3: $\frac{a_2}{a_1} = 2.995$.

v_1	Tap Number.									
	1	2	3	4	5	6	7	8	9	10
5	−0.025	−0.025	−0.025	+0.04	+0.12	+0.20	+0.21	....	.. .	+0.21
10	−0.05	−0.063	−0.042	+0.22	+0.50	+0.60	+0.63	....	....	+0.55
15	−0.07	−0.095	−0.053	+0.68	+1.24	+1.50	+1.50	....	....	+1.43
20	−0.08	−0.120	−0.051	+1.14	+2.26	+2.66	+2.75	....	....	+2.57
25	−0.07	−0.130	−0.025	+1.75	+3.36	+4.02	+4.15	....	....	+3.83
30 (from curves produced)	−0.06	−0.140	0.	+2.42	+4.53	+5.32	+5.62	....	....	+5.21

TABLE 15.—Hydraulic Gradients.

Combination No. 4: $\frac{a_2}{a_1} = 6.46$.

r_1	Tap Number.						
	1	2	3	4	5	6 and 7	9 and 10
5.....	—0.05	—0.05	—0.05	+0.01	+0.08	+0.10	+0.10
10.....	—0.09	—0.103	—0.10	+0.02	+0.20	+0.25	+0.25
15.....	—0.12	—0.157	—0.15	+0.10	+0.52	+0.62	+0.58
20.....	—0.17	—0.21	—0.20	+0.30	+1.23	+1.40	+1.35
25.....	—0.18	—0.26	—0.24	+0.60	+2.03	+2.30	+2.25
30.....	.....	—0.10	—0.26	+0.94	+2.89	+3.26	+3.20

TABLE 16.—Hydraulic Gradients.

Combination No. 5: $\frac{a_2}{a_1} = 9.32$.

r_1	Tap Number.								
	1	2	3	4	5	6	7	8	9
5.....	—0.05	—0.55	—0.053	—0.045	+0.02	+0.04	+0.07	+0.07	+0.07
10.....	—0.106	—0.12	—0.12	—0.09	+0.11	+0.18	+0.20	+0.20	+0.20
15.....	—0.155	—0.173	—0.175	—0.125	+0.32	+0.50	+0.53	+0.53	+0.51
20.....	—0.197	—0.222	—0.225	—0.121	+0.70	+1.03	+1.11	+1.11	+1.10
25.....	—0.212	—0.258	—0.258	—0.076	+1.16	+1.72	+1.77	+1.77	+1.75
30.....	—0.199	—0.260	—0.250	—0.012	+1.75	+2.46	+2.48	+2.48	+2.47

Fig. 14 shows a set of gradients for a common velocity and different area ratios. The friction effect has been corrected by increasing the ordinates by an amount equal to the computed friction drop from Tap 0, or the reference pressure tap. It will be noted that the maximum change in pressure occurs when the area ratio is about 1 to 2 or 1 to 3, which is what would be expected from the theory as deduced by the application of the laws of maxima and minima.

Distance of Maximum p_2 from Plane of Enlargement.

It was readily seen that this distance (call it x) would undoubtedly depend on the thickness $(r_2 - r_1)$ of the water between the stream from the smaller pipe, and the inside surface of the large pipe, or would vary as some function of $(d_2 - d_1)$.

For each of the combinations tested the value of x was determined within limits from the gradients and from inspection of the data. These limits were plotted on logarithmic paper with $(d_2 - d_1)$ in

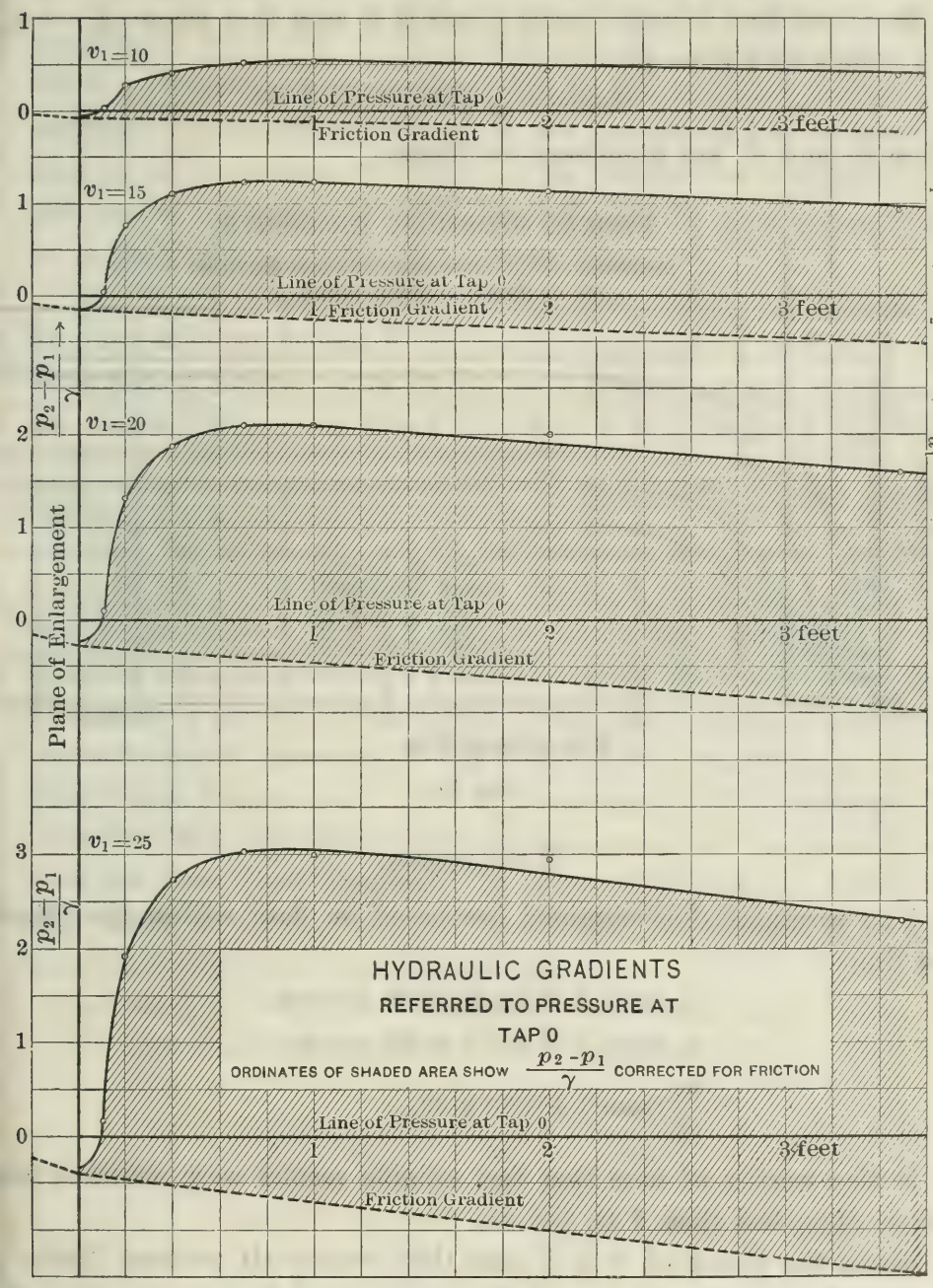

$v_1 = 10$
Line of Pressure at Tap 0
Friction Gradient
1
2
3 feet
$v_1 = 15$
Line of Pressure at Tap 0
Friction Gradient
$v_1 = 20$
Line of Pressure at Tap 0
Friction Gradient
$v_1 = 25$
Line of Pressure at Tap 0
Friction Gradient
Plane of Enlargement
$\frac{p_2 - p_1}{\gamma}$
HYDRAULIC GRADIENTS
REFERRED TO PRESSURE AT
TAP 0
ORDINATES OF SHADED AREA SHOW $\frac{p_2 - p_1}{\gamma}$ CORRECTED FOR FRICTION
$d_1 = .486$; $d_2 = .994$; $\frac{a_2}{a_1} = .451$

FIG. 13.

inches as abscissas. It was found that a straight line with slope = 0.4 would pass very nearly through the average values, which would indicate that $x = K\ (d_2 - d_1)^{0.4}$.

From the line on logarithmic paper it is seen that when $(d_2 - d_1) = 1$, then $x = 1.45 = K$.

Hence $x = 1.45\ (d_2 - d_1)^{0.4}$

where d_2 and d_1 are diameters, in inches.

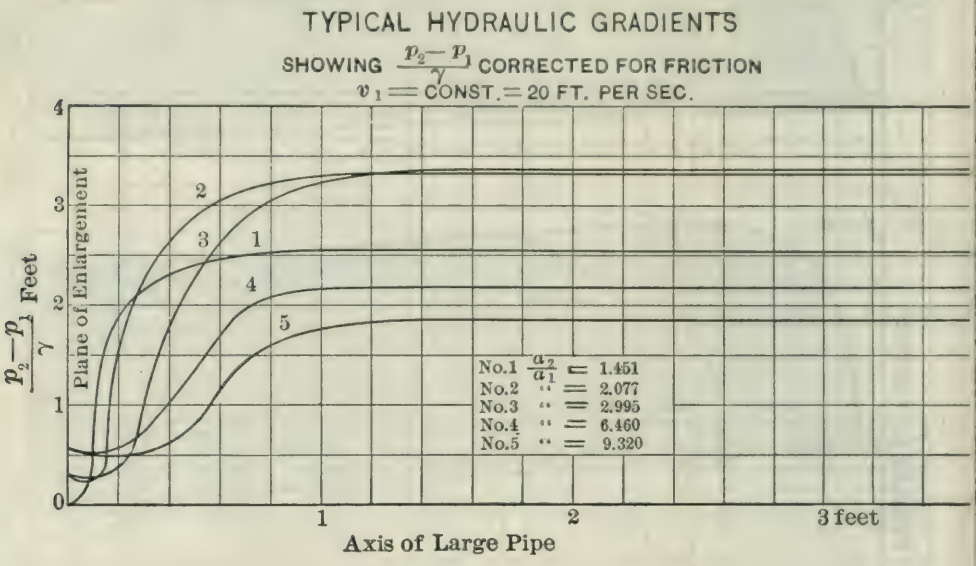

TYPICAL HYDRAULIC GRADIENTS
SHOWING $\frac{p_2 - p_1}{\gamma}$ CORRECTED FOR FRICTION
v_1 = CONST. = 20 FT. PER SEC.
$\frac{p_2 - p_1}{\gamma}$ Feet
Plane of Enlargement
No.1 $\frac{a_2}{a_1}$ = 1.451
No.2 " = 2.077
No.3 " = 2.995
No.4 " = 6.460
No.5 " = 9.320
Axis of Large Pipe
3 feet

FIG. 14.

CONCLUSIONS.

The formulas thus obtained are based on data within the following limits:

v_1 from 3.82 to 31.58 ft. per sec.

v_2 from 0.07 to 11.44 ft. per sec.

$\frac{a_2}{a_1}$ from 1.451 to 9.32.

It is also worthy of note that few reliable results were obtained with values of v_1 less than 6 ft. per sec.

From the values of B it is seen that, within all ordinary limits of values of velocity and area ratios, the actual loss is less than the theoretical loss. This, at first, seems peculiar, but it is well known that velocity head may be changed to pressure head, with more or less attendant loss, by using a diverging passage to decrease the velocity

gradually and cause as little disturbance of stream lines as possible. In practice, this is accomplished by diffusers made of solid material. In the present instance, such action occurs with the dead water at the sides of the stream entering with velocity, v_1, having such diffusing effect. This explanation is borne out by the fact that the value of B decreases, or the amount by which the theoretical loss is lessened increases, as the velocity increases, or as the relative "deadness" of the side water increases. B also decreases as the area ratio increases, or as the quantity of dead water increases.

Mr. A. H. Gibson* deduces the same theoretical loss as given herein, and substantiates it with some experimental work he had done. Such results seem to agree closely with the theory, but, unfortunately, the maximum, p_2, was taken too close to the plane of enlargement, resulting in a smaller value of $\frac{p_2 - p_1}{\gamma}$ and hence a larger lost head. He used the value of $x = 10$ in., which, according to the formula, $x = 1.45\ (d_2 - d_1)^{0.4}$ should have been 1.7 ft. or 20.4 in. for maximum p_2, as $(d_2 - d_1) = 1.5$ in. The statement is made that by taking as p_2 the value obtained at a point 4 in. from the enlargement, the loss is about 20% greater than the theoretical, which agrees closely with the results herewith presented, as the actual loss decreases as p_2 increases, other things being equal. Had p_2 been taken at its maximum, losses from 0 to 20% less than theoretical, depending on the velocity, would doubtless have been observed. It is probable that the values of p_2 were taken at about the point where the loss changes from greater than theoretical to less than theoretical.

Summary of Results.—The summary of results is as follows:

(1) Lost head due to sudden enlargement $= L.H.$

$$L.H. = 1.098 \frac{v_1^{1.919}}{2g} \left(1 - \frac{a_1}{a_2}\right)^{1.919}$$

$$\text{or } L.H. = 1.098 \frac{(v_1 - v_2)^{1.919}}{2g}$$

$$\text{or } L.H. = \frac{B\ (v_1 - v_2)^2}{2g}, \text{ where } B \text{ is a variable coefficient}$$

with values given in Table 11.

* "Hydraulics and Its Applications," published in 1908.

(2) Distance of maximum p_2 from plane of enlargement $= x$

$$x = 1.45\ (d_2 - d_1)^{0.4}$$

where d_2 and d_1 are diameters in inches.

The writer takes this opportunity to thank all who helped bring this work to completion. The experimental work was done under the general supervision of Professor J. N. Le Conte, who gave many helpful suggestions in the computations as well. H. T. Cory, M. Am. Soc. C. E., also gave much valuable advice in connection with the coordination of the results and their preparation for presentation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1263

ON LONG-TIME TESTS OF PORTLAND CEMENT.*

By I. Hiroi, M. Am. Soc. C. E.

With Discussion by Messrs. Chandler Davis, L. J. Le Conte, George G. Honness, and George S. Binckley.

The behavior of Portland cement in sea water has long been a subject of discussion by chemists and engineers; yet the lack of complete experimental data, on one hand, and vast amounts of material and labor placed at stake in most harbor works of magnitude relying on the durability of cement, on the other, seem to make any results of investigation on its nature a matter of ever recurrent interest, especially to those engaged on maritime works. It is under such an impression that the writer presents some of the results of his still unfinished experiments, the completion of which, however, he may not live to see.

More than 15 years ago, at a time when the writer had to give out instructions for manufacturing concrete blocks for use in harbor works, he commenced a series of tests on the behavior of the cements then in use, in order to ascertain as far as possible the reliability of the rules laid down for the execution of the works. The tests have included the quantity and quality of water used in gauging, the quantity and quality of the sand and ballast, the differences of temperature, the kind of surroundings, the modes of fabrication, and, in short, most of the important points in the manufacture of mortar and concrete considered likely to affect the strength of finished work in course of time. Most of these tests have confirmed the correctness

* Presented at the meeting of February 19th, 1913.

of the method as found in treatises relating to the subject and generally practiced in good works, so that it is hardly worth while describing here the results obtained. The durability test alone seems to be of some interest, particularly to those who have doubts as to the permanence of cement structures built in sea water.

The tests have been proposed to extend over 50 years, and although the time which has elapsed hardly covers one-third of that period, the results thus far obtained are of some value in showing the general trend of changes taking place in the strength of cement, and in forecasting in a certain measure the eventual results of the tests.

Three different kinds of cement and one hydraulic lime have been used for the various tests. In this paper only the results obtained with two kinds of cement will be considered. The chemical analyses of these two cements are given in Table 1.

TABLE 1.—Chemical Analyses of Cements.

	Cements.	
	A	*B*
Loss by combustion	0.88	1.24
Insoluble matter	1.30	1.25
Silica	22.20	19.00
Alumina	7.55	8.35
Ferric oxide	2.90	3.90
Lime	61.77	62.17
Magnesia	1.12	1.75
Miscellaneous	2.28	2.34

In quality, the cements satisfied the following requirements:

The cement shall be so fine that the residue on a sieve of 5 000 meshes per sq. in. will not exceed 10% of the whole.

The insoluble matter contained in the cement shall not exceed 3 per cent.

The cement shall not commence to set within 1 hour of mixing with water.

The cement in setting shall not show any change in shape or volume.

Briquettes composed of 1 part of cement and 3 parts of standard sand, and immersed in sea water, shall have a tensile strength of not less than 130 lb. per sq. in. after 1 week, and gradually increasing to 170 lb. in 4 weeks, the amount of that increase to be not less than 30 lb. per sq. in.

All the briquettes for the durability test were made by hand, the mortar being gauged with fresh water and struck into the mould (with a trowel weighing $1\frac{3}{4}$ lb.) until water rose to the surface. The briquettes thus made were placed in a covered box for 24 hours, after which they were transferred to several media. The sands used for the mortar were of two kinds: (1) Coarse beach sand, with grains of all sizes up to about 0.06 in.; and (2) standard sand obtained by sifting the coarse sand between sieves with 400 and 1 400 meshes per sq. in. The briquettes kept in air were placed in an open shed, and those in water in tanks in which the water was changed once a week. The briquettes are uniformly 1 sq. in. in section, and five of them have been taken out for each single test.

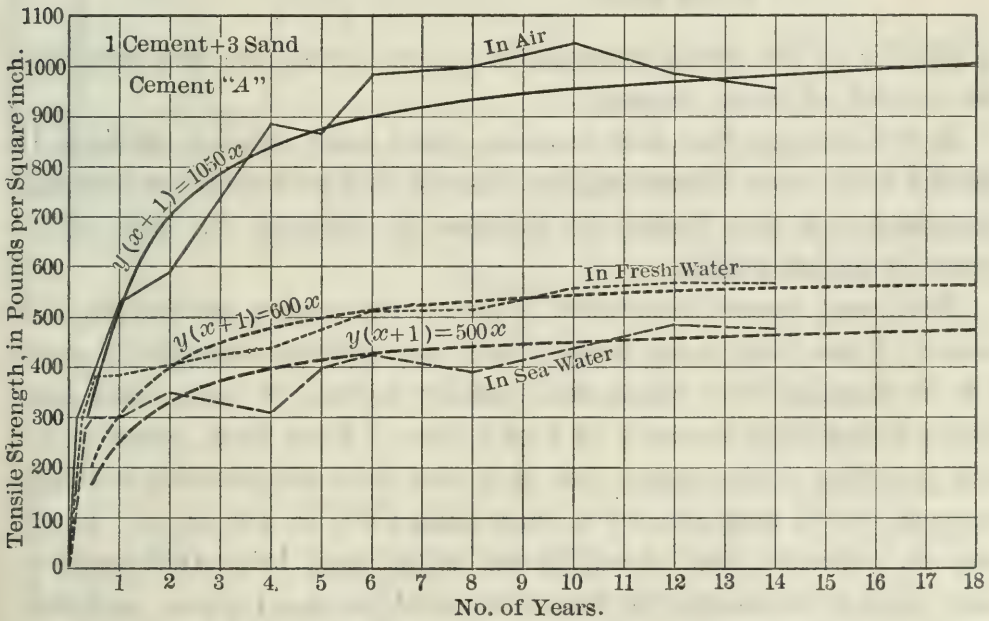


FIG. 1.

Fig. 1 shows the results of tests made with a mortar of 1 cement and 3 sand, by weight. These briquettes were placed separately in air, fresh water, and sea water, 24 hours after fabrication, in the manner already described, and remained there until tested. The strengths of briquettes made with coarse and standard sands have been found to be so nearly alike that the mean results were taken in plotting the curves. The strength attained by the use of finer sand naturally came out considerably lower. It will be seen, as might have been anticipated, that the mortar attains the greatest strength in air

and the least in sea water, the mean comparative ratio of strength attained being nearly as follows:

Air	1.00
Fresh water	0.56
Sea water	0.45

The curves of strength thus traced are approximately hyperbolic, and may be more or less closely expressed by the following equations:

$$\text{In air} \ldots\ldots\ldots\ldots\ldots y = \frac{1\,050\,x}{x + 1}$$

$$\text{In fresh water} \ldots\ldots\ldots y = \frac{600\,x}{x + 1}$$

$$\text{In sea water} \ldots\ldots\ldots\ldots y = \frac{500\,x}{x + 1}$$

in which $y =$ the tensile strength, in pounds per square inch, and $x =$ the number of years elapsed.

It will be seen that such mortars attain their greatest strength in about 6 to 10 years, beyond which, although they still continue (through occasional ups and downs) to increase in strength, the rate of increase is inconsiderable.

With neat cement briquettes (Fig. 2), the results are entirely different. Those kept in sea water attain their greatest strength in about 2 to 10 months, after which they rapidly decline, in some cases completely losing their strength in 4 or 5 years. Even then, however, not only are their forms intact, but they also show considerable crushing strength, which may amount to more than 6 000 lb. per sq. in. In the case of hydraulic lime placed in sea water, neat briquettes began to show signs of disintegration from the outside in about a year, and after about 4 years even the sound core, which had been decreasing in size all that time, finally succumbed, leaving a shapeless mass. In air and fresh water, neat briquettes continue to increase in strength for 4 or 5 years, when they attain much higher strength than those placed in sea water. A comparison of the air curves on Figs. 1 and 2 shows the decided superiority, in the long run, of the use of 1 cement + 3 sand mortar over neat cement. The total loss of tensile strength in neat cement briquettes, whatever may be the medium in which they are placed, appears to be a question of time.

Fig. 3 shows another series of tests made with briquettes which had been kept immersed in fresh water for 2 months before they were finally

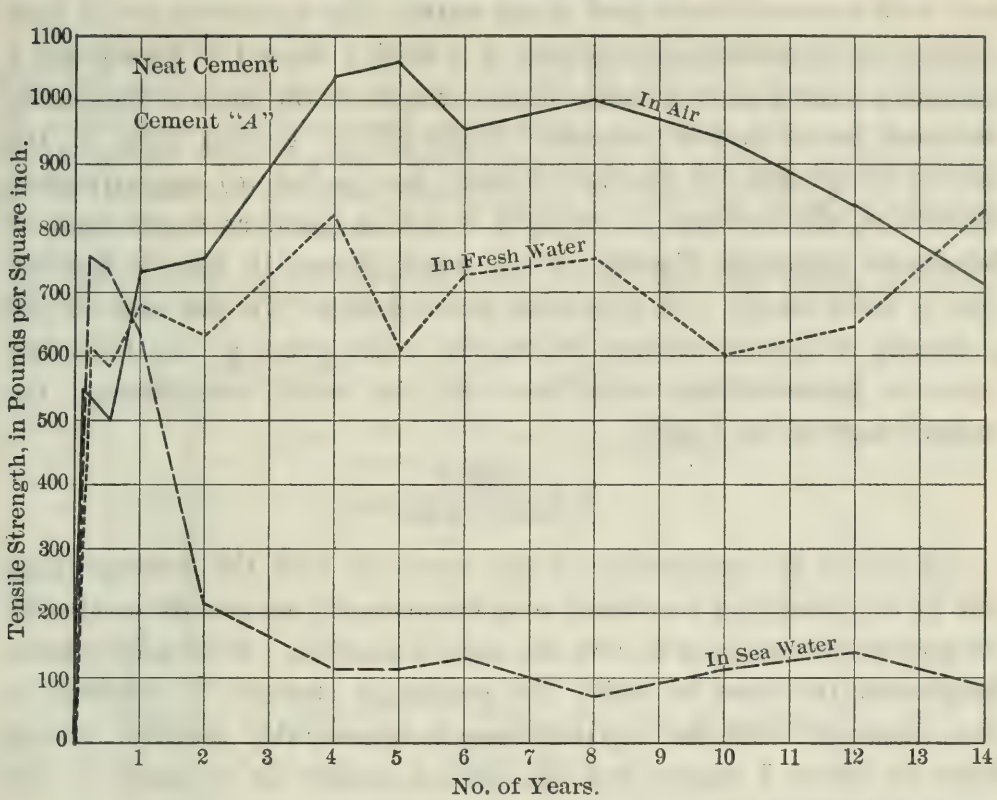

Neat Cement
Cement "A"
In Air
In Fresh Water
In Sea Water
Tensile Strength, in Pounds per Square inch.
No. of Years.

FIG. 2.

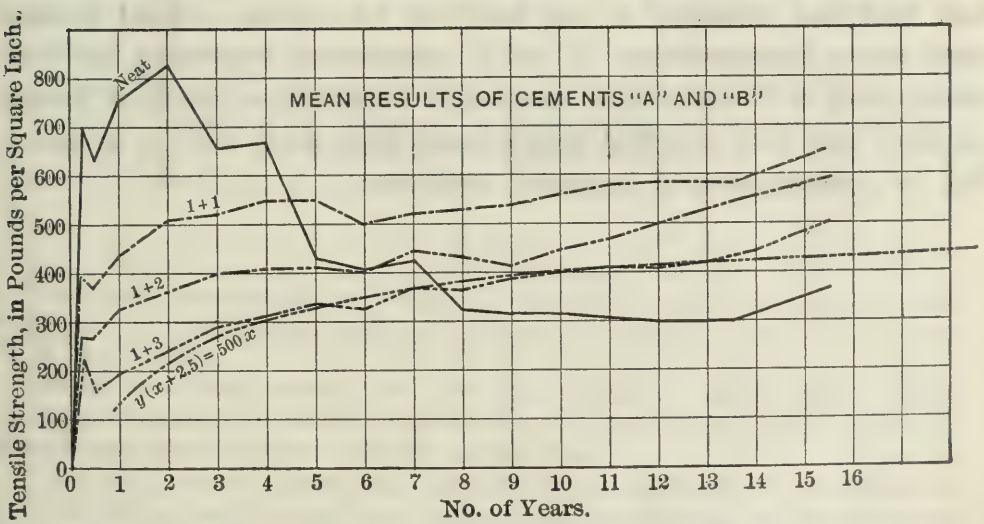

MEAN RESULTS OF CEMENTS "A" AND "B"
Neat
1+1
1+2
1+3
y(x+2.5)=500x
Tensile Strength, in Pounds per Square Inch.
No. of Years.

FIG. 3.

placed in sea water in partial reproduction of the conditions which obtain with concrete blocks used in sea water. The briquettes are of neat cement, and of mortars of 1 cement + 1 sand, 1 cement + 2 sand, and 1 cement + 3 sand, all by weight. Each curve shows the mean of the results obtained for *A* and *B* cements. Although the general form of the curves, except that for the neat cement, are (as before) approximately hyperbolic, the increase of strength is not as rapid as in the case of briquettes immersed directly in sea water, shown in Fig. 1, but the rise is more steady and continues much longer. In the case of the 1 cement + 3 sand mortar, the regular curve given by the following equation approximately coincides with the actual one through the greater part of its length:

$$y = \frac{500\,x}{x + 2.5}.$$

As far as the comparison of this equation with the corresponding one in the preceding case bears out, the strength apparently attainable by such mortar appears to have the same limitation. With neat cement briquettes, the time in which the maximum strength is attained in this, compared with the previous case, is considerably retarded, taking place in about 2 years; and the rate of decline in strength is also much less rapid.

Although the results of these tests are not yet conclusive as to the permanence of strength of cements, their indications are that, with proper selection of materials, right proportions of ingredients, and with due attention to the mode of fabrication, cement mortars used under circumstances in which engineering structures are commonly built, will continue to increase in strength for indefinite lengths of time, and thus artificial stones made with them will be as enduring as natural ones of approved qualities.

DISCUSSION

Mr. Davis.

CHANDLER DAVIS, M. AM. SOC. C. E. (by letter).—While connected with the Dock Department of the City of New York, the writer carried on a series of long-time tests of Portland cements, covering a period of 10 years. The cements used conformed to all the requirements of the Department standard specifications of 1898, the principal clauses of which were as follows:

"Slow-setting cement, gauged neat, must not set within $\frac{1}{2}$ hour after being mixed with water.

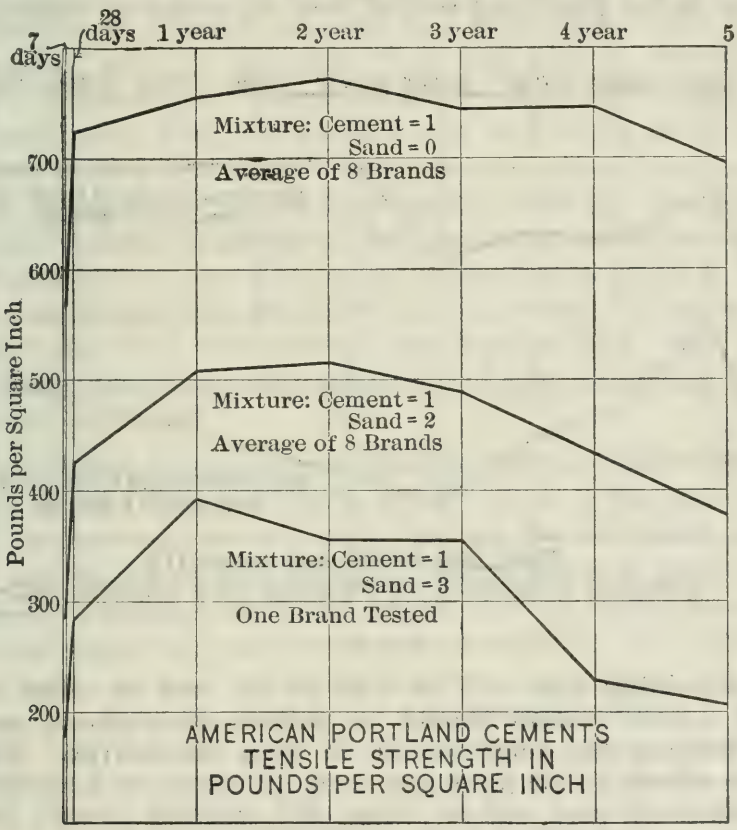


FIG. 4.

"The neat cement, after 7 days (1 day in air and 6 in water) must be capable of sustaining without rupture, a tensile strain of at least 300 lb. per sq. in.

"Mixed one part cement to two parts sand, it must, after 7 days (1 day in air and 6 in water), be capable of sustaining without rupture, a tensile strain of at least 125 lb. per sq. in.

"The neat cement, when set, must show no distortions or cracks.

"The time of setting will be determined by a wire $\frac{1}{24}$ in. in diameter, loaded so as to weigh 1 lb. The cement will be considered set when this wire is supported by the cement without showing any indentation.

Mr. Davis.

"The tests for checking or cracking and for color will be made by moulding on thin plates of glass, three cakes of neat cement, 2 or 3 in. in diameter, and about ½ in. thick in the center, and with very thin edges. One of these cakes, when it is set perfectly hard, will be put in water and examined from day to day to see if it leaves the glass, becomes distorted, or if cracks show themselves on the thin edges. Such distortions or cracks indicate that the cement is unfit for use at that time.

"The second of these cakes will be allowed to set hard in moist air, at a temperature of about 200°, for about 3 hours. It will then be placed in boiling water, and remain in boiling water for a period of from 24 to 48 hours, and, at the end of that time, the cake should still adhere to the glass, and should show no cracks or distortions.

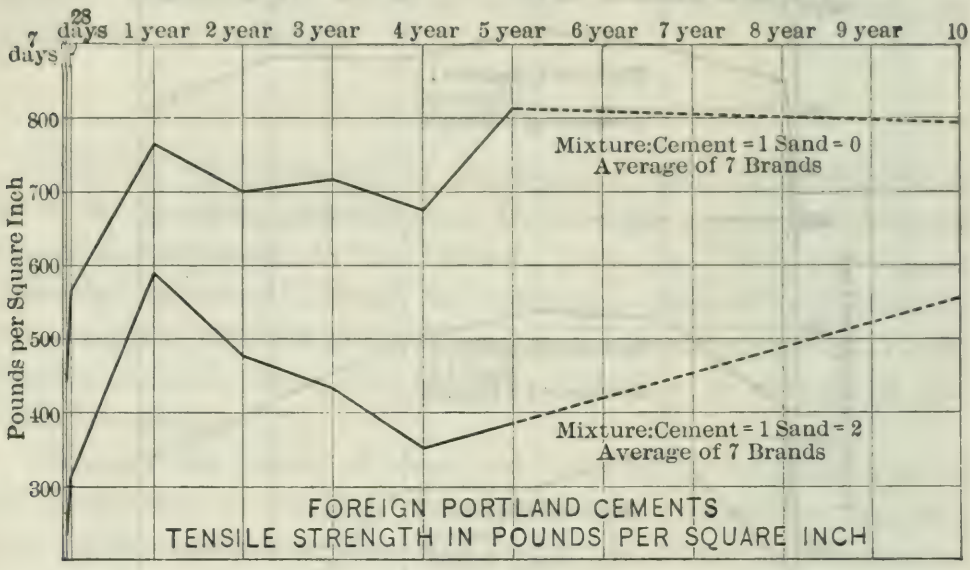


FIG. 5.

"The remaining cake will be kept in air, and its color observed, which for a good cement should be uniform throughout, and for a Portland cement, the color should be of a bluish-gray. This cake should also adhere to the glass and show no cracks or distortions.

"The standard sand will be clean, dry, crushed quartz, passing a No. 20 sieve and caught on a No. 30 sieve.

"Ordinary, fresh, clean water, having a temperature between 60 and 70° Fahr., will be used for the water mixture and immersion of samples."

All the tests were for tensile strength, and the results obtained have been plotted on Figs. 4 and 5.

The curves shown on Fig. 4 are the average strength curves of several brands of American Portland cement. The resulting curves are very irregular. The cement seems to have acquired its maximum strength at the end of the second year and then gradually decreased in strength until, after a lapse of 5 years, the tensile strength is less

than it was at the end of 28 days. The tests of the American brands were only continued for a period of 5 years. Mr. Davis.

The average results of seven brands of foreign cements are plotted on Fig. 5. Eliminating the 7- and 28-day tests, it will be seen that the neat cement reached its maximum strength at the end of 5 years, after having passed through a minimum at the end of 4 years. The strength of the cement at the end of the tenth year is only 17 lb. less than it was at the end of the fifth year. The strength curve for a mixture of one part cement and two parts sand, if the 7- and 28-day tests are eliminated, shows a maximum at the end of the first year, again a minimum at the end of the fourth year, with a continuation of the increase in strength through the fifth year, until the tenth year is reached, when it was only 37 lb. less than at the end of the first year. Unluckily, no samples were made for breaking between the fifth and tenth year, so no idea can be formed as to how the cement acted during that time.

All the briquettes were moulded and broken by Mr. Robert M. Sterritt, who has been in charge of the Department Testing Laboratory since its organization, thus eliminating errors due to personal equation.

It is to be hoped that Mr. Hiroi will continue his series of very interesting tests, and will publish the results from time to time, simultaneously describing the condition of the structures in which these cements were used.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is very much interested in this instructive paper. It is to be regretted, however, that the author does not state that all the briquettes were well protected against the hurtful effects of severe frosts in winter, though it is natural to presume that they were. Mr. Le Conte.

The experiments on sand briquettes are certainly very encouraging. The experiments on neat cement briquettes in sea water, on the contrary, are very discouraging, from a theoretical point of view. In view of the fact that the great German cement expert, Dr. Michaelis, has clearly proved that the disintegration of Portland cement in sea water is due to the chemical action of such water on the ordinary constituents of the cement, it is natural to suppose that some effect is to be expected, in the natural course of events—it being purely a question of time in any case.

Taking the worst case reported: If it can safely be assumed that the penetration of chemical action into the neat cement briquette is proportional to the loss in strength in a given time, then it amounts to approximately 0.32 in. in 4 years, and a tensile strength of 100 lb.; then, 10 years after, or 14 in all, we still have the same tensile strength, 100 lb., showing that chemical action had reached its extreme limit of penetration, namely, 0.32 in. There is certainly nothing very startling

Mr. Le Conte.

about this result, and, moreover, the concrete in all harbor works is necessarily exposed to marine growths of all kinds, such as barnacles, mussels, oysters, shell-fish, and all kinds of sea-weeds and grasses, which soon form a heavy coating over the entire water face of all the concrete work, and practically cut off all free circulation of sea water. Hence the writer does not think that the interesting results reported furnish any information which calls for serious consideration; on the contrary, the sand briquettes stand up well under the 14-year tests, much better, in fact, than one would naturally expect, theoretically.

The probabilities are that the penetration of chemical action into the sand briquettes, which undoubtedly does exist, is so very small in amount as to be completely obliterated by the general slow growth in strength. The falling off in strength of neat cement briquettes between the ages of 5 and 14 years, when kept in ordinary air, is one of the mysteries yet to be explained. It seems to be the experience everywhere. It naturally follows from the above that low-grade porous concrete may have its strength seriously affected by the chemical action of sea water.

Mr. Honness.

GEORGE G. HONNESS, M. AM. SOC. C. E. (by letter).—Actual construction seldom affords opportunity to make long-time tests of cement, consequently, such information, though valuable, particularly in this age of concrete and the growing use of reinforced concrete, should be secured by tests made by manufacturers, laboratories of technical schools, or municipal departments having the direction of engineering works.

The construction of the New Croton Dam afforded an opportunity to make long-time tests, and the writer, though not connected with that work, had access to the records, and believes them of sufficient value and interest to publish.

These records afford no opportunity to compare Mr. Hiroi's results in respect to conditions which obtain when the cement is used in salt water, but indicate the behavior of the cement and mortar under normal conditions. One brand of Portland and three brands of natural or Rosendale cement were used in this work, during the progress of which the methods of manufacture changed from the dome kiln to the rotary kiln. Practically all the testing was done by one man, which fact eliminates any inconsistency due to personal manipulation. The mortar briquettes were composed of 1 part cement to 2 parts standard sand, instead of the 1:3 mixture now prescribed; the standard sand was a crushed quartz, all of which would pass a No. 20 and be retained on a No. 30 sieve.

Table 2 shows the results of the tests, and it will be noted that they represent the average of a large number of briquettes. The dia-

Mr. Honness.

TABLE 2.—TESTS OF PORTLAND AND NATURAL CEMENTS USED IN NEW CROTON DAM, 1894-1905.

Age.	Tensile Strength, in Pounds per Square Inch.															
	Portland, Neat, No. 1.		Portland, 1:2 Mortar, No. 1.		Natural, Neat, No. 2.		Natural, 1:2 Mortar, No. 2.		Rosendale, Neat, No. 3.		Rosendale, 1:2 Mortar, No. 3.		Rosendale, Neat, No. 4.		Rosendale, 1:2 Mortar, No. 4.	
	No. of Briquettes.	Average.	No. of Briquettes.	Average.	No. of Briquettes.	Average.	No. of Briquettes	Average.	No. of Briquettes.	Average.	No. of Briquettes.	Average.	No. of Briquettes.	Average.	No. of Briquettes.	Average.
1 Day.....	14 610	217	.....	....	8 461	108	.....	....	12 265	64	.....	...	527	68	...	...
1 Week.....	14 740	547	12 216	397	8 449	159	7 308	89	12 294	118	8 994	41	551	142	551	34
1 Month.....	1 985	601	1 938	529	1 000	235	979	190	1 079	220	1 070	139	85	192	80	114
3 Months.....	1 398	669	1 378	677	815	361	785	342	565	374	580	270	75	392	75	232
6 Months.....	1 018	664	1 011	727	665	424	667	441	480	403	510	333	56	393	58	298
9 Months.....	832	658	797	748	628	450	624	489	395	402	390	342	40	342	46	341
1 Year.....	825	667	815	769	612	460	615	527	445	375	460	327	58	279	55	299
1¼ Years.. ...	686	654	671	779	570	471	558	550	420	381	400	330	37	301	32	344
1½ Years.....	508	650	513	769	485	474	470	570	355	404	355	346	35	334	25	360
2 Years.....	404	647	412	753	327	476	317	584	380	399	380	351	25	325	20	352
2½ Years.....	367	646	384	756	352	453	335	575	325	386	300	348	25	267	20	362
3 Years.....	370	697	335	755	277	443	255	600	290	394	275	335	15	300	15	363
4 Years.....	197	674	205	784	183	459	190	635	235	409	185	356	15	290	15	398
5 Years.....	164	669	171	751	150	466	155	697	220	423	232	328	20	304	20	406
6 Years.....	27	651	90	684	22	482	35	604	105	416	90	319	...	...	...	...
7 Years.....	95	658	73	695	61	494	83	729	55	414	54	337	...	...	...	...
8 Years.....	71	624	85	739	79	516	104	745	.....	...	.....	...	...	...	...	...
9 Years.....	35	635	55	644	26	428	47	718	.....	...	.....	...	...	...	...	...
10 Years.....	33	639	18	611	50	500	49	732	.....	...	.....	...	...	...	...	...

Riehle Bros. 1 000-lb. testing machine used.
Cement made by dome kiln method up to 1903; afterward by rotary kiln.
Briquettes up to 3 years broken by maker. Of the 4 to 10-year briquettes, 87% broken by maker.

Mr. Honness.

gram, Fig. 6, shows the results of the tests graphically. These tests show some interesting features, the chief of which is that the natural cement mortars show a gradual increase of strength to the end of the period, and the Portland cement shows retrogression of strength, after 3 years in neat cement and after 4 years in 1:2 mortar. Another interesting feature is that after 4½ years the natural 1:2 mortar exceeds the strength of the Portland No. 1. This confirms the belief of some of the older engineers, that the lasting qualities of natural cement are superior to those of native Portlands.

The retrogression of strength shown by these tests, although it represents only one brand of Portland cement, makes one ask: Is this fall-

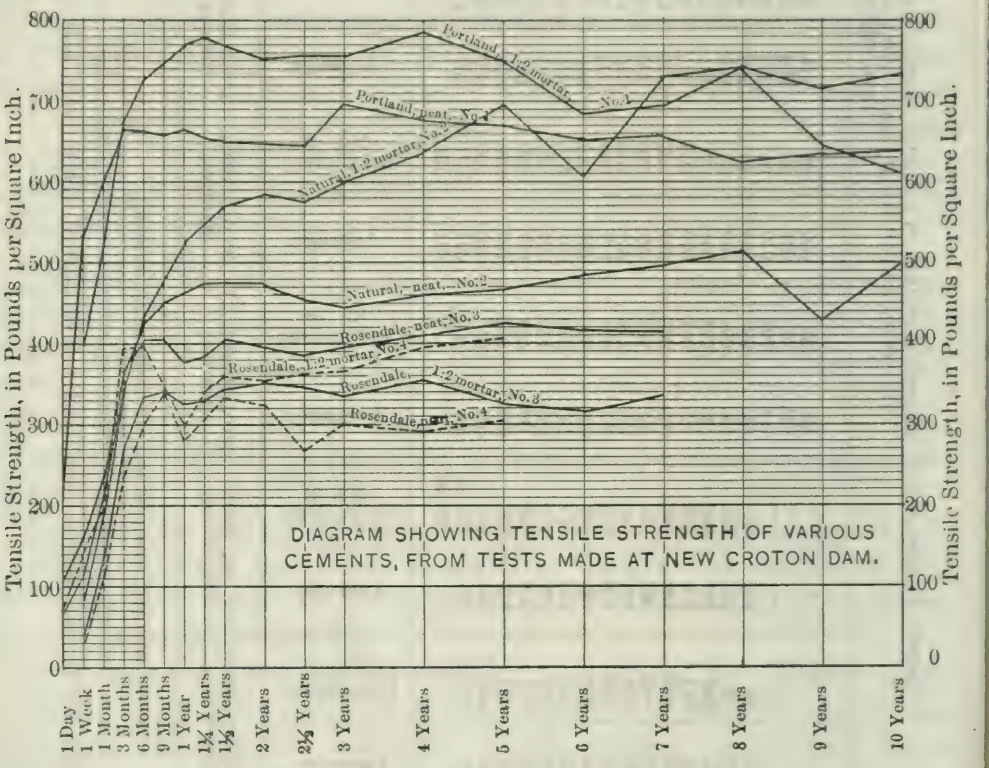


FIG. 6.

ing off of strength continuous, or when does it cease? This is of especial importance when considered in connection with reinforced concrete.

The author's experiments indicate that the brands of cement used by him continued to gain in strength after an age of 10 years. It may be that these brands have the proper combination of ingredients to assure a continual increase of strength, or at least no serious retrogression. Unfortunately, it is impossible for the writer to give the chemical composition of the Portland cement used at the New Croton Dam for comparison with the brands used by the author.

Mr. Honness.

It seems desirable that, from data of this kind now available or to become available, a determination should be made of the chemical combination in cement which would assure that the strength would not fall below a fixed minimum after a long period of years. When the proper combination has been determined, it should be embodied in all specifications for important masonry construction.

Mr. Binckley.

GEORGE S. BINCKLEY, M. AM. SOC. C. E. (by letter).—In providing a stimulating center of discussion, Mr. Hiroi's brief paper on long-time tests of cement is particularly valuable. Such data as those he presents are certainly of high importance, and the curves given by Mr. Davis, Figs. 4 and 5, are still more suggestive, especially in consideration of the conditions under which these really startling results have been obtained.

These two sets of curves present such a striking contrast between the American and foreign cements as to call for more than passing comment. They are of peculiar interest to the writer in tending to strengthen a growing suspicion that American cements lack the inherent soundness found in the best of those of foreign manufacture.

Some years ago, the writer had occasion to make a comparison between one of the best-known of the English Portland cements and a cement made in Western Canada, and in the course of these tests some very curious and suggestive results were obtained. So striking were some of these that the writer used them as the basis for a brief article published soon afterward.* In few words, these particular tests had to do with the qualities of the coarser particles in Portland cement—those passing the 100-mesh and remaining on the 200-mesh screen.

In the cements examined, this material, contrary to the commonly accepted idea, showed very considerable hydraulic properties; in fact, briquettes gave a tensile strength at 7 days of 126 lb. for the Canadian, and 220 lb. for the English briquettes; at 28 days the figures were 215 lb. for the Canadian and 353 lb. for the English briquettes. Mr. Spackman, of Philadelphia, has suggested to the writer that part of this effect was due to fine particles adhering to the coarser ones, yet he admits a certain hydraulicity in at least the superficial part of the coarse particles themselves.

Both these cements passed the standard tests successfully, the gain in strength between 7 and 28 days being 90 lb. in each. Both cements, neat, stood the accelerated tests successfully; and the Canadian cement showed a greater strength than the English at both 7 and 28 days.

When the briquettes or pats made from this intermediate material—that passing the 100-mesh and remaining on the 200-mesh screen—

* *Engineering Record*, Feb. 19th, 1910, p. 212.

Mr. Binckley.

were subjected to the steam test under atmospheric pressure, however, the difference in results was startling. The accompanying photographs of these test pats and briquettes (Figs. 7 and 8) make this very clear. The English pat and broken briquette remained perfectly hard and sound, and those made from the intermediate material of the Canadian cement quickly broke down. To quote from the article previously referred to:

"Standard boiling pats were prepared and placed on the same glass plate to secure perfect identity of conditions. On account of the crumbling (noted above) of the edges of the briquettes made from the intermediate material screened out of the Canadian cement, these boiling pats were kept in moist air 60 hours before boiling to insure a perfectly fair test.

"The two pats representing the two cements investigated were then placed in cold water, which was raised to the boiling point. Boiling was continued for 2½ hours, at the end of which time the condition of the two pats may be seen by reference to the accompanying photograph. That pat made from the intermediate material of the English cement showed no deterioration whatever, adhered tightly to the glass, and was free from any sign of cracking. It in every respect indicated perfect soundness. The other pat, made from intermediate material screened out of the sample of Canadian cement, showed the first sign of breaking down 45 minutes after being placed in the water, and its integrity was completely destroyed in two hours and a half. Its condition is most graphically shown in the accompanying photograph, in which it is seen that it is cracked and disintegrated to such an extent that its interior is in a granular or sandy condition.

"These results were so striking that a further boiling experiment was made on half of one of the briquettes, the 28-day [Canadian] specimen showing a tensile strength of 215 lb. per square inch. * * * This briquette was boiled when it was 38 days old. Yet in spite of its comparatively high strength and thorough seasoning in and out of water, it broke down completely after about five hours' boiling, as shown in the photograph.

"This complete failure was a surprise, as there had been no visible sign of deterioration before boiling; yet but a short time after the water had reached the boiling point the first sign of distress appeared."

The writer considered these results highly suggestive. Here are two cements, each apparently conforming to the standard tests for soundness; yet, when one is subjected to the writer's test, it is seen to contain an element of weakness, and the other remains sound. The Canadian cement seems to contain about 20% by weight of a material inherently mischievous, but protected in such a way, under the standard tests, as to remain undiscovered. In other words, this Canadian cement—and probably a great many American cements also—contains in its coarser particles an element of weakness which evades discovery under the standard tests, but may reasonably be expected to reveal itself as the years pass. This is strikingly indicated in Mr. Davis'

FIG. 7.—"PATS OF INTERMEDIATE MATERIAL AFTER BOILING."

FIG. 8.—"BRIQUETTE OF INTERMEDIATE MATERIAL."

curves, and, in the case of neat cements, in the curves presented by Mr. Honness (Fig. 6), and also, to a less striking yet notable degree, in Mr. Hiroi's paper. Mr. Binckley.

Though it would hardly be proper to consider the characteristics of American cements as indicated by these data as a cause of actual alarm, they can hardly be considered satisfactory. Even qualified and at least partly compensated by the better showing made by the sand briquettes, Mr. Davis shows curves representing an average of eight brands of American cement that lead one to wonder where the rapid drop in strength will stop, if at all. It must be remembered, also, that those eight brands probably represent the best cements obtainable for the works considered, as it is incredible that inferior or suspected cements would be placed under a long-time test and allowed to drag down a good average. The curve Mr. Davis gives for a single brand in 1:3 sand briquettes is rather appalling, for it has lost 50% in 4 years, and is still falling off.

The engineer cannot afford to contemplate with composure the possibility of a progressive deterioration of a supposedly permanent material of such importance. If cement is to fall under the head of materials subject to deterioration, the sooner we know it the better, to the end that its life may be calculated, replacements provided for, and means taken to prevent disastrous failures. If, on the other hand, we are to have a dependable material for all the ages, which, built into our structures will actually—as so confidently assumed—improve in strength with the years, defects must be sought out, and faulty material must be rejected without mercy. In the article previously referred to, the writer suggested the advisability of applying the test described as a ready means for the detection of inherent weakness in a cement passing the standard tests. Since that time, a still more drastic test—the autoclave test—has been advocated, but this requires special and rather costly apparatus, and is so drastic that it seems probable that few American cements would succeed in passing it. Yet the writer believes that the time will come very soon when engineers will find it prudent or even imperative to subject the cement required for works of importance to far more searching tests than those now in use; and if the cement manufacturers find it difficult or even impossible to supply cement of the required integrity under their present methods of manufacture, they will simply have to improve their methods. As soon as the certainty of eventual weakness in concrete made with any particular class of cement is established, or even a strong suspicion of concrete as a permanent material is aroused, some very strict requirements will have to be met by the cement manufacturers.

Mr. Binckley.

In the meantime, the writer suggests—even urges in cases of importance—the application of the tensile and steam tests to that part of the material passing the 100-mesh and remaining on the 200-mesh screen. This material should be made into standard boiling pats and briquettes, and carefully observed. Everything else being apparently equal, as between rival cements, this test will certainly give a definite indication of relative soundness that should fix one as superior.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1264

EXPERIMENTS ON WEIR DISCHARGE.

By W. G. Steward,* Esq., and J. S. Longwell,† Jun. Am. Soc. S. E.

With Discussion by Messrs. C. H. Pierce, Francis F. Longley, W. G. Steward and J. S. Longwell.

The experiments discussed in this paper were made by the writers at the experiment station of the Boise Project of the United States Reclamation Service during 1911, in connection with duty of water investigations for the project. The project experiment station is on the main canal of the Boise Project, near Boise River, at a point about 8 miles above Boise, Idaho, and was built in the fall of 1910.

The water used for this station is diverted into a forebay from the main canal by a double turn-out consisting of two 18-in. concrete pipes, each controlled separately by gates on the bank of the main canal, as shown by Fig. 1. The forebay or weir pool is 30 ft. square on the bottom, 4 ft. deep, and has side slopes of 1½ to 1. The bottom of this bay is 12 ft. below the bottom of the main canal. An adjustable baffle-board divides the forebay into two parts. The sides of the pool above the baffle are rip-rapped with large stones in order to prevent the water from washing the banks as it issues from the turn-out. Below the baffle the water is quiet, and no rip-rap is needed.

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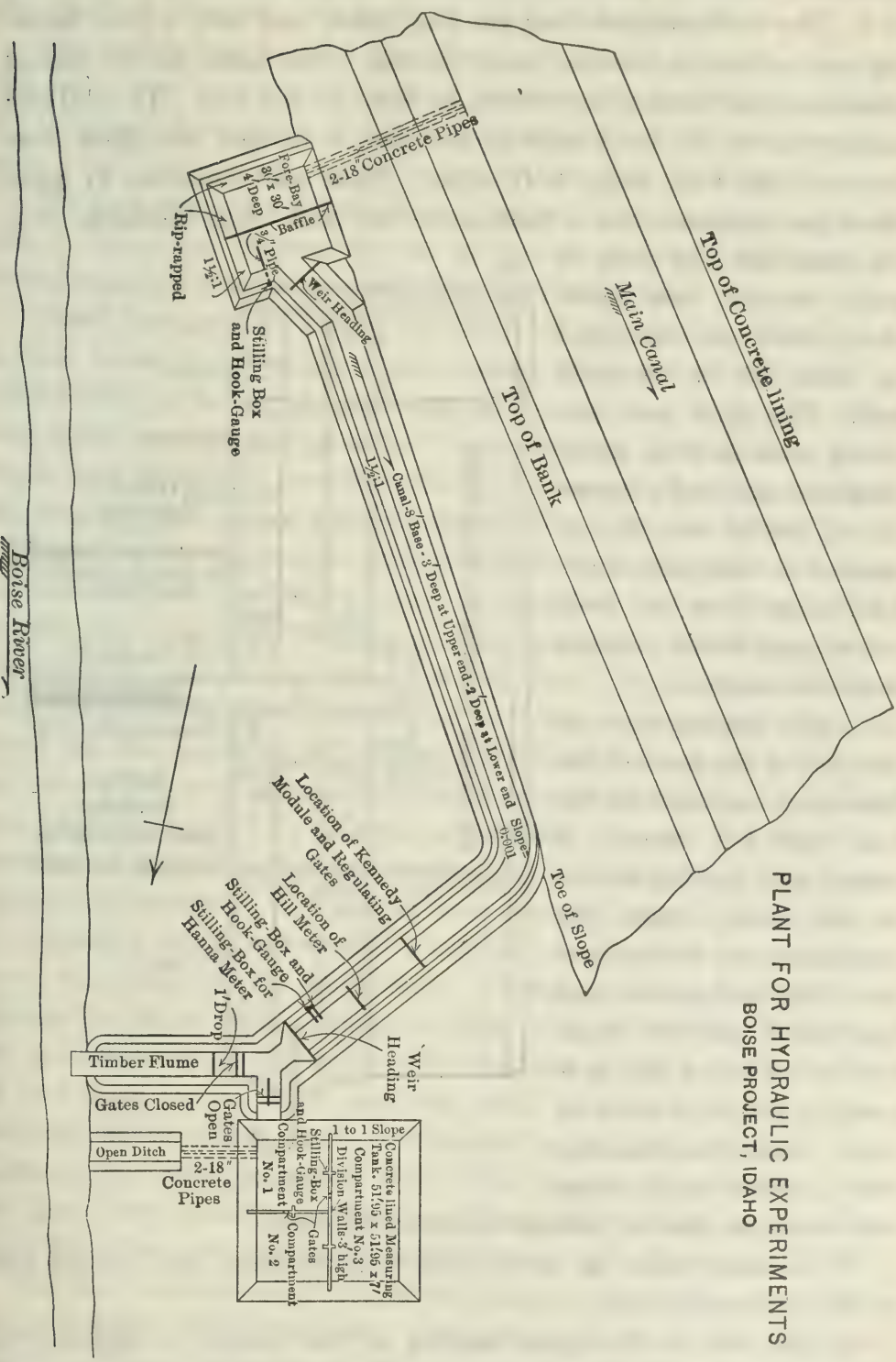
A canal having a bottom width of 8 ft. carries the water about 318 ft. from this forebay to the upper end of a timber flume, 8 ft. wide and 3 ft. deep. This flume section extends for 12 ft. and then divides into two sections of equal width at right angles with each other and located so that the center line of each makes an angle of 45° with the center line of the main flume. The left branch leads to a concrete measuring tank; the right branch leads to Boise River, and acts as a wasteway. A set of gates in each branch controls the flow of water, diverting it into the tank or into the river, as may be desired. There are two gates in each set, a large one hinged to a vertical post set just enough off center to cause the water pressure to keep it closed, and a small water-tight one hinged to the floor and held in a vertical position by a hinged brace fastened to one of the cross-pieces on top of the flume. A water-proof canvas is nailed to the sides and floor of the flume and to this gate, with enough extra material in the ends, so that the gate can be operated without difficulty. The large gate serves to stop most of the flow, thus making it easier to operate the small one. The closing of one set of gates requires from 1 to 2 sec. There was no difficulty in operating the gates, and they proved very satisfactory and efficient.

This gate heading was caulked and coated with a tar preparation, and no leakage could be detected, either through the flume or the gates. The upper portion of the flume, between the gates and the weir, acted as a storage basin for the water between the time of closing one set and the opening of the other. The weir was set high enough above the flume floor so that the water which backed up between the shutting and opening of the gates did not cause any submergence on the weirs. In the case where the weirs were set at the upper heading, the flume and canal acted as the storage basin, and there was no effect on the weir discharge.

In obtaining the intervals for the various runs, the time was recorded at the closing of the gates in the wasteway at the beginning of the run and the closing of the opposite set at the end of the experiment. Thus the only possible error that might enter into the results would be the difference in times required for closing, which at the most was about $\frac{1}{2}$ sec.

The concrete measuring tank is set into the ground 7 ft. The



PLANT FOR HYDRAULIC EXPERIMENTS
BOISE PROJECT, IDAHO

bottom is 51.95 ft. square, and the side slopes are on a batter of 1 to 1. The walls and the floor are 6 in. thick, and have a ½-in. finishing coat which is troweled very smooth. The walls are in perfect alignment, and present no uneven surfaces of any sort. To facilitate measurements, the lower part of the tank is divided into three compartments by 6-in. walls, 3 ft. high. These are connected by gates which are operated from a walk above the tank. The discharge from the tank into the river is made through two 18-in. pipes which are controlled by gates set in the tank wall. The tank was calibrated accurately by measurements, and had a capacity of 24 000 cu. ft. A number of tests were made for leakage from the tank, but no appreciable quantity could be detected.

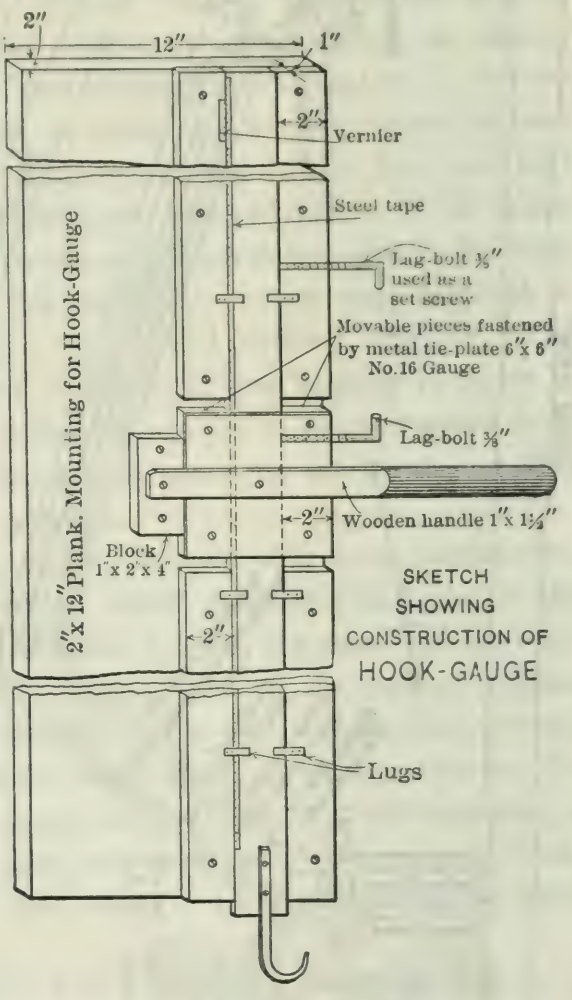


FIG. 2.

A weir heading was constructed at the head of the flume and was used for the 6-in. and 1-ft. weirs. A second weir heading was set at the point where the canal takes out of the forebay. This heading was used for the 2-ft. and 3-ft. weirs, and was of such a size as to permit a 6-ft. weir to be set on it. Both these headings were caulked with oakum and tarred, so that no leakage whatever occurred.

Hook-gauges were set above each weir heading, and a third one in the measuring tank.

In the case of the upper heading at the forebay, a stilling-box, 1 ft. square, was set back into the bank, with a ¾-in. pipe leading out into the forebay about 15 ft. above the weir. The hook-gauge was

attached to the inside of the stilling-box, and the head on the weirs was observed above the effect of the velocity of approach.

The stilling-box for the lower weir at the flume heading was of the same size as above; it was set back in the bank of the canal, 10 ft. above the weir, and was connected with the canal by a $\frac{3}{4}$-in. pipe. The hook-gauge was set in the manner just described.

The hook-gauge used in the measuring tank was placed in a stilling-box, 1 ft. square, fastened to one of the division wall buttresses in Compartment No. 1 and operated from the plank walk.

These hook-gauges, Fig. 2, consisted of a 1 by 2-in. strip with a hook fastened to the lower end, sliding between two similar 1 by 2-in. strips. A length of steel tape, reading to hundredths of a foot, was tacked on this sliding strip. A vernier was screwed on a small block, and the block was then nailed to one of the strips so that the vernier edge almost touched the steel tape on the sliding strip. This enabled the observer to read accurately to thousandths of a foot. In order to hold the slide in place, a set-screw was attached to one of the strips, and, by turning it, the slide was forced into close contact with the opposite strip and held firm. At a position on the gauge about 3 ft. above the ground or walk on which the observer stood, the two nailed strips were cut away for a distance of 6 in., and in their place two strips, considerably shorter than those removed, and fastened together by a metal plate, were inserted. A lever was fastened to a block nailed to the plank holding the gauge at a point opposite the center of the opening, and was also fastened to the metal of the small double slide with a screw. A set-screw was attached to the outer strip of the double slide. Thus, by turning this set-screw against the single slide holding the steel tape and forcing it against the opposite strip of the double slide, the single slide could be moved up and down by the lever, the hook could be brought to the proper position, and the slide was then clamped by the first set-screw. The reading could then be taken. This lever enabled the observer to set the gauge very quickly and also very accurately. These gauges were made by the writers, and served their purpose very effectively.

The weir experiments consisted of calibrations of 6-in., 1-ft., 2-ft., and 3-ft. sharp-crested Cippoletti and rectangular weirs with full contraction. The experiments on sharp-crested Cippoletti and rectangular weirs were made, not with the idea of obtaining new general formulas

for weirs, but for the purpose of testing the general weir formulas and ascertaining whether they are applicable to the weirs on the Boise and other projects of the Reclamation Service, and, if necessary, of preparing rating tables for the weirs tested and a formula or table by which accurate discharges might be obtained for a range of weirs and heads above the limits of previous experiments.

The crests of all the weirs were set about 1 ft. above the canal bottom. The 6-in. and 1-ft. weirs were located in the center of the 8-ft. canal at the flume heading. The 2-ft. and 3-ft. weirs were located at the head of the canal, and took the water directly from the 30-ft. forebay. The gauges were set far enough above the weirs to be free from the effect of velocity of approach. At the flume heading, baffles were placed in the canal just below a regulating gate, and planks were laid on the water to do away with any surface disturbances. A very even and uniform flow resulted, with no appreciable velocity of approach except in one or two high heads on the 1-ft. Cippoletti weir. In the case of the weirs used on the upper heading, the regulation was obtained with the gates in the main canal. Very little trouble was experienced after the water had settled in the pool. The water in the main canal was from 4 to 5 ft. in depth over the gates, and any slight variation in head did not affect the discharge of the pipes appreciably. The hook-gauges above the weirs were read every 30 sec. throughout the length of runs, which varied from 5 min. to 1 hour, depending on the head. The average of the readings gave the gauge height for each particular run. The different compartments of the tank were used as the discharge increased. For the low heads and smaller weirs Compartment No. 1 was used. As the head increased, Compartments Nos. 1 and 2 were used, and, for the large heads, the entire tank was utilized. No trouble was experienced in reading the gauges to thousandths of a foot.

Cippoletti Weirs.—The experiments on Cippoletti weirs are divided into five parts: (1) calibrations of the 6-in. weir; (2) an attempt to determine the shape of the 6-in. weir to discharge according to the Cippoletti formula; (3) calibrations of the 1-ft. weir; (4) calibrations of the 2-ft. weir; and (5) calibrations of the 3-ft. weir.

In the 6-in. weir, twenty-four runs were made with heads ranging from 0.161 to 0.945 ft. The maximum variation in gauge height during any run was 0.024 ft. No appreciable velocity of approach

TABLE 1.—Results of Experiments on Weirs.

Cippoletti Weirs.

Series No.	Run No.	Average gauge height.	Variation in gauge height.	Length of weir, in inches.	Time interval, in seconds.	Actual discharge.	Discharge, in second-feet.	C in formula, $Q = Clh^{\frac{3}{2}}$
		h		l			Q	
1	1	0.217	0.004	6	4 560	781	0.171	3.383
	2	0.225	0.000	6	5 100	913	0.179	3.352
	3	0.334	0.024	6	2 970	978	0.329	3.408
	4	0.336	0.002	6	3 935	1 365	0.347	3.562
	5	0.334	0.002	6	2 490	855	0.343	3.552
	6	0.513	0.004	6	2 620	1 822	0.695	3.783
	7	0.667	0.001	6	1 625	1 894	1.166	4.284
	8	0.770	0.005	6	1 365	1 905	1.396	4.132
	9	0.941	0.000	6	880	1 739	1.976	4.329
	10	0.839	0.012	6	1 075	1 773	1.649	4.291
	11	0.776	0.017	6	1 255	1 788	1.424	4.166
	12	0.675	0.000	6	1 255	1 402	1.117	4.028
	13	0.537	0.000	6	1 392	1 059	0.761	3.867
	14	0.940	0.009	6	970	1 918	1.977	4.338
	15	0.945	0.005	6	900	1 793	1.992	4.337
	16	0.774	0.001	6	1 380	1 939	1.405	4.126
	17	0.715	0.000	6	1 165	1 428	1.226	4.055
	18	0.600	0.000	6	945	856	0.906	3.898
	19	0.400	0.000	6	990	460	0.465	3.676
	20	0.830	0.000	6	1 200	1 897	1.581	4.181
	21	0.667	0.000	6	1 200	1 292	1.077	3.954
	22	0.530	0.000	6	1 110	813	0.732	3.795
	23	0.270	0.000	6	1 260	307	0.244	3.478
	24	0.161	0.004	6	780	83	0.106	3.287
7	1	0.637	0.007	12	600	1 064	1.773	3.487
	2	0.594	0.010	12	480	759	1.581	3.454
	3	0.588	0.005	12	480	744	1.550	3.437
	4	0.518	0.001	12	480	609	1.269	3.404
	5	0.482	0.002	12	600	687	1.145	3.422
	6	0.409	0.002	12	600	533	0.888	3.394
	7	0.302	0.001	12	600	335	0.558	3.361
	8	0.215	0.002	12	1 200	397	0.331	3.320
	9	0.138	0.001	12	1 200	205	0.171	3.333
	10	0.840	0.019	12	510	1 413	2.771	3.599
	11	0.846	0.002	12	600	1 683	2.805	3.604
	12	0.659	0.006	12	540	1 017	1.883	3.520
	13	0.693	0.002	12	600	1 222	2.037	3.530
	14	0.815	0.004	12	600	1 587	2.645	3.595
	15	0.927	0.004	12	600	1 958	3.263	3.656
	16	0.992	0.003	12	600	2 189	3.648	3.692
	17	1.097	0.003	12	600	2 591	4.318	3.758
	18	1.219	0.003	12	600	3 097	5.162	3.835
	19	1.320	0.003	12	480	2 849	5.935	3.913
	20	1.386	0.002	12	360	2 333	6.481	3.971
	21	1.476	0.004	12	360	2 604	7.233	4.034
	22	.....	.....	..	...	.....	.....	.....
	23	1.528	0.001	12	360	2 771	7.697	4.076
	24	1.604	0.000	12	360	3 014	8.372	4.121
	25	1.721	0.008	12	300	2 836	9.453	4.186
	26	1.853	0.008	12	300	3 227	10.757	4.265
	27	1.897	0.002	12	270	3 042	11.267	4.312
	28	1.978	0.009	12	240	2 915	12.145	4.366
8	1	0.066	0.003	24	1 200	146	0.122	3.588
	2	0.186	0.002	24	1 200	662	0.552	3.441
	3	0.317	0.000	24	600	727	1.212	3.392
	4	0.318	0.002	24	600	727	1.212	3.380
	5	0.430	0.002	24	600	1 146	1.910	3.386
	6	0.438	0.003	24	600	1 172	1.953	3.368
	7	0.526	0.001	24	600	1 550	2.583	3.385
	8	0.524	0.001	24	600	1 541	2.568	3.385
	9	0.601	0.002	24	600	1 890	3.150	3.380

TABLE 1.—(*Continued.*)

Series No.	Run No.	Average gauge height.	Variation in gauge height.	Length of weir, in inches.	Time interval, in seconds.	Actual discharge.	Discharge in second-feet.	C in formula, $Q = Ch^{\frac{3}{2}}$.
		h		l			Q	
8	10	0.594	0.004	24	600	1 859	3.098	3.381
	11	0.588	0.001	24	600	1 824	3.040	3.371
	12	0.668	0.003	24	600	2 235	3.725	3.411
	13	0.679	0.003	24	600	2 286	3.810	3.197
	14	0.853	0.004	24	600	3 257	5.428	3.445
	15	0.854	0.004	24	600	3 266	5.443	3.448
	16	1.086	0.005	24	420	3 227	7.921	3.499
	17	1.090	0.003	24	420	3 359	7.998	3.514
	18	1.238	0.006	24	330	3 234	9.800	3.557
	19	1.484	0.011	24	600	7 871	13.118	3.628
	20	1.101	0.016	24	600	4 844	8.073	3.494
	21	1.425	0.010	24	600	7 350	12.250	3.601
	22	1.537	0.006	24	600	8 247	13·745	3.607
	23	1.598	0.007	24	630	9 391	14.906	3.689
	24	1.816	0.017	24	540	9 822	18.189	3.719
	25	1.852	0.016	24	480	9 004	18.758	3.722
	26	1.987	0.010	24	660	13 981	21.092	3.737
	27	0.406	0.004	24	630	1 108	1.727	3.338
	28	0.290	0.003	24	900	949	1.054	3.374
	29	0.178	0.001	24	900	461	0.512	3.409
	30	0.154	0.003	24	1 200	502	0.418	3.455
	31	0.092	0.002	24	1 800	347	0.193	3.459
	32	0.059	0.002	24	1 800	201	0.112	3.889
	33	0.034	0.001	24	3 660	173	0.047	3.730
11	1	0.172	0.002	36	1 200	883	0.736	3.436
	2	0.170	0.002	36	1 200	867	0.722	3.433
	3	0.082	0.004	36	3 600	909	0.252	3.574
	4	0.236	0.002	36	900	1 068	1.187	3.450
	5	0.302	0.000	36	600	1 025	1.708	3.430
	6	0.609	0.004	36	600	2 920	4.867	3.414
	7	0.500	0.001	36	600	2 164	3.607	3.400
	8	0.460	0.006	36	480	1 521	3.169	3.386
	9	0.297	0.002	36	600	989	1.648	3.393
	10	0.130	0.002	36	900	438	0.487	3.461
	11	0.090	0.001	36	1 260	354	0.281	3.469
	12	0.216	0.001	36	900	924	1.027	3.410
	13	0.444	0.002	36	600	1 793	2.988	3.366
	14	0.598	0.003	36	600	2 831	4.718	3.400
	15	0.844	0.005	36	600	4 764	7.940	3.413
	16	0.994	0.003	36	600	6 146	10.243	3.445
	17	1.167	0.005	36	600	7 796	12.993	3.435
	18	1.410	0.007	36	360	6 262	17.394	3.463
	19	0.396	0.002	36	600	1 506	2.510	3.357
	20	1.470	0.005	36	420	7 836	18.657	3.489
	21	1.923	0.006	36	300	8 404	28.013	3.502
	22	.....	.....	..	...	.....	.....	.....
	23	2.415	0.009	36	300	12 416	41.387	3.676
	24	2.246	0.008	36	240	8 560	35.667	3.532
	25	2.065	0.008	36	360	9 418	31.393	3.526

existed during this experiment, and the end contractions and the shape of the nappe were not disturbed in any manner. Compartment No. 1 of the measuring tank was used entirely during the calibrations of this weir.

Plate XXXIV shows the gauge height and discharge curve, and Table 2 gives the actual discharge for each $\frac{1}{100}$ ft. of gauge height. Plate XXXVI is a logarithmic discharge curve from which it will be

PLATE XXXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1264.
STEWARD AND LONGWELL ON
WEIR DISCHARGE EXPERIMENTS.

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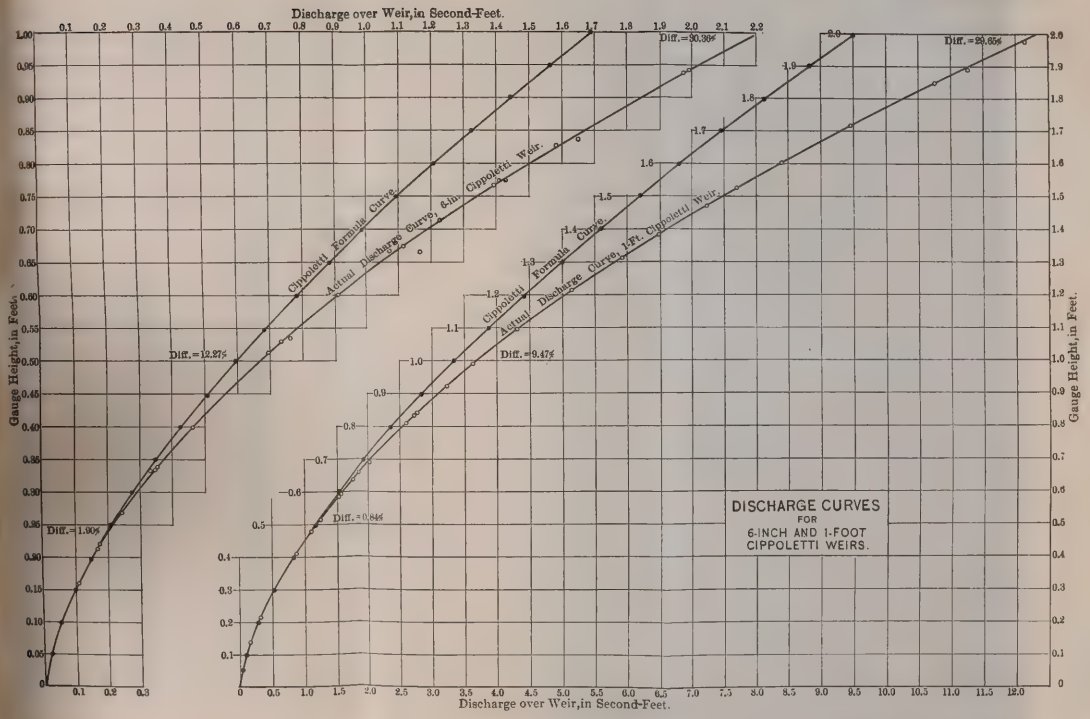


seen that three distinct parabolas exist as the discharge curve, instead of one. This logarithmic curve was used in determining the proper plotting of the gauge height discharge curve. The equations of the different parabolas were determined as shown in Table 7, also the gauge height at which the change of equation takes place, and the proportional part that this gauge height is of the crest length.

TABLE 2.—Gauge Height and Discharge for 6-Inch Cippoletti Weir.

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.00			0.51		0.691	
0.01		0.002		0.52		0.713	
0.02		0.005		0.53		0.735	
0.03		0.009		0.54		0.758	
0.04		0.013		0.55		0.781	
0.05	0.019	0.019		0.56		0.805	
0.06		0.025c		0.57		0.830	
0.07		0.031		0.58		0.855	
0.08		0.038		0.59		0.880	
0.09		0.045		0.60	0.782	0.906	15.86
0.10	0.053	0.053		0.61		0.932	
0.11		0.061		0.62		0.959	
0.12		0.070		0.63		0.987	
0.13		0.079		0.64		1.015	
0.14		0.088		0.65		1.043	
0.15		0.098		0.66		1.071	
0.16		0.108		0.67		1.099	
0.17		0.118		0.68		1.128	
0.18		0.129		0.69		1.157	
0.19		0.139		0.70	0.986	1.186	20.28
0.20	0.151	0.151		0.71		1.215	
0.21		0.162		0.72		1.245	
0.22		0.174		0.73		1.275	
0.23		0.186		0.74		1.305	
0.24		0.200		0.75		1.336	
0.25		0.214	1.90	0.76		1.367	
0.26		0.228		0.77		1.398	
0.27		0.243		0.78		1.429	
0.28		0.258		0.79		1.460	
0.29		0.273		0.80	1.205	1.492	23.82
0.30	0.277	0.288	3.97	0.81		1.524	
0.31		0.304		0.82		1.557	
0.32		0.320		0.83		1.590	
0.33		0.337		0.84		1.623	
0.34		0.354		0.85		1.657	
0.35		0.371		0.86		1.691	
0.36		0.389		0.87		1.726	
0.37		0.407		0.88		1.761	
0.38		0.425		0.89		1.796	
0.39		0.444		0.90	1.437	1.832	27.49
0.40	0.426	0.463	8.69	0.91		1.868	
0.41		0.482		0.92		1.904	
0.42		0.502		0.93		1.940	
0.43		0.522		0.94		1.976	
0.44		0.542		0.95		2.012	
0.45	0.508	0.563		0.96		2.048	
0.46		0.584		0.97		2.084	
0.47		0.605		0.98		2.120	
0.48		0.626		0.99		2.157	
0.49		0.647		1.00	1.683	2.194	30.36
0.50	0.595	0.669	12.45				

A number of runs were made to determine the shape of a weir that would discharge according to the Cippoletti formula. Plates were screwed on the face of a 6-in. Cippoletti weir, and a general shape was determined. The shape obtained, Fig. 3, was a series of reverse curves or waves, and seems to be of no practical value.

The 1-ft. Cippoletti weir was placed on the flume heading, and the flow was regulated by special gates. The gauge heights on this weir ranged from 0.138 to 1.978 ft., with a maximum variation of gauge height of 0.019 ft. for any run. No corrections were made for velocity of approach, as the maximum correction was only 0.002 ft. for the highest head of 1.978 ft. over the weir. The effect on the lower heads was not appreciable. In all cases there was full contraction and a free nappe. A gauge height discharge curve, also shown on Plate XXXIV, was plotted with the assistance of a logarithmic curve, as described for the 6-in. weir. Table 3 gives the discharge for each $\frac{1}{100}$ ft. of gauge height. Compartment No. 1 in the tank was used for the lower heads, and Nos. 1 and 2 for the higher ones.

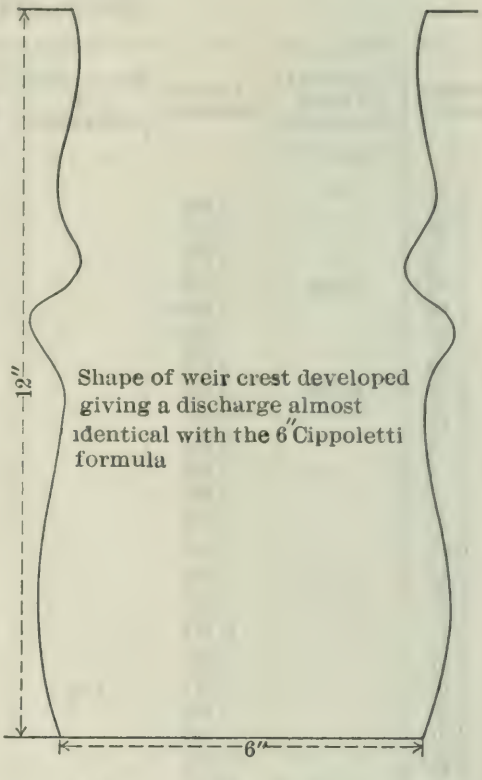
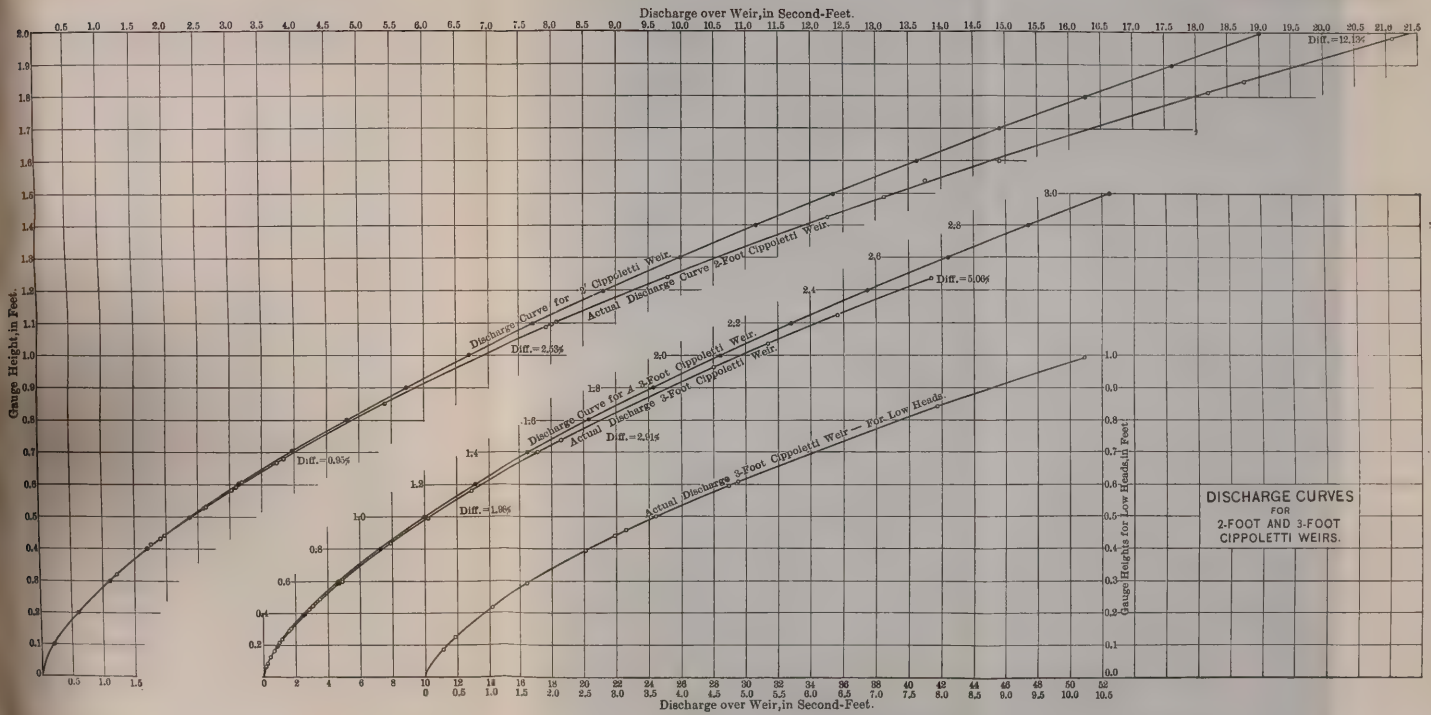


FIG. 3.

The 2-ft. and 3-ft. Cippoletti weirs were set at the head of the canal, taking the water from the 30-ft. forebay. No velocity of approach existed, as the gauge was set to give the elevation of the water in the forebay above the effect of the weirs. For the 2-ft. weirs, the heads ranged from 0.034 to 1.987 ft., with a maximum variation of head of 0.017 ft. for any run. For the 3-ft. weir, the heads varied from 0.082 to 2.415 ft. The greatest variation of head was 0.009 ft. The contraction in both weirs was entirely full, and the nappes were perfectly free from any disturbances. Compartment No. 1 of the measuring tank was used for the low heads, Nos. 1 and 2 for interme-

1054.


Discharge over Weir, in Second-Feet.
Gauge Height, in Feet.
Discharge Curve for 2' Cippoletti Weir.
Actual Discharge Curve 2-Foot Cippoletti Weir.
Diff. = 2.53%
Diff. = 0.95%
Diff. = 12.13%
Discharge Curve for a 3-Foot Cippoletti Weir.
Actual Discharge 3-Foot Cippoletti Weir.
Diff. = 2.01%
Diff. = 1.98%
Diff. = 5.00%
Actual Discharge 3-Foot Cippoletti Weir — For Low Heads.
Gauge Heights for Low Heads, in Feet.
DISCHARGE CURVES
FOR
2-FOOT AND 3-FOOT
CIPPOLETTI WEIRS.
Discharge over Weir, in Second-Feet.

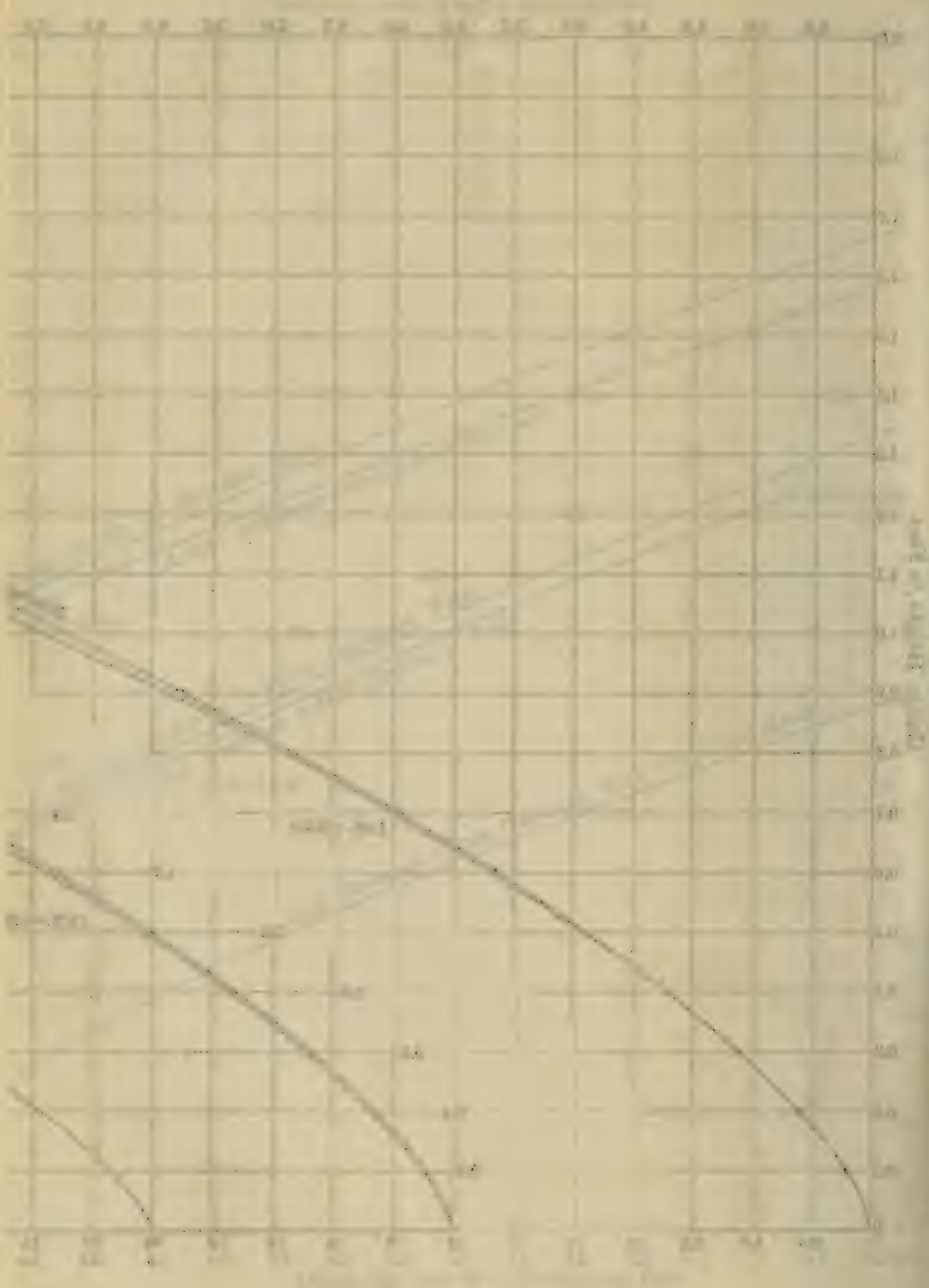


TABLE 3.—Gauge Height and Discharge for 1-Foot Cippoletti Weir.

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.000	0.000		0.66		1.871	
0.01		0.003		0.67		1.919	
0.02		0.010		0.68		1.968	
0.03		0.018		0.69		2.017	
0.04		0.027		0.70	1.972	2.066	4.77
0.05	0.038	0.038		0.71		2.115	
0.06		0.050		0.72		2.164	
0.07		0.062		0.73		2.213	
0.08		0.076		0.74		2.262	
0.09		0.091		0.75		2.312	
0.10	0.107	0.107		0.76		2.362	
0.11		0.123		0.77		2.412	
0.12		0.140		0.78		2.462	
0.13		0.158		0.79		2.513	
0.14		0.176		0.80	2.409	2.565	6.48
0.15		0.196		0.81		2.618	
0.16		0.216		0.82		2.671	
0.17		0.236		0.83		2.724	
0.18		0.257		0.84		2.777	
0.19		0.279		0.85		2.830	
0.20	0.301	0.301		0.86		2.884	
0.21		0.324		0.87		2.939	
0.22		0.347		0.88		2.994	
0.23		0.371		0.89		3.049	
0.24		0.396		0.90	2.875	3.105	8.00
0.25		0.421		0.91		3.161	
0.26		0.446		0.92		3.217	
0.27		0.472		0.93		3.274	
0.28		0.499		0.94		3.331	
0.29		0.526		0.95		3.389	
0.30	0.553	0.553		0.96		3.447	
0.31		0.581		0.97		3.506	
0.32		0.609		0.98		3.565	
0.33		0.638		0.99		3.625	
0.34		0.667		1.00	3.367	3.686	9.47
0.35		0.697		1.01		3.747	
0.36		0.727		1.02		3.809	
0.37		0.758		1.03		3.872	
0.38		0.789		1.04		3.935	
0.39		0.820		1.05		3.999	
0.40	0.852	0.852		1.06		4.063	
0.41		0.884		1.07		4.127	
0.42		0.916		1.08		4.192	
0.43		0.949		1.09		4.258	
0.44		0.983		1.10	3.884	4.325	11.35
0.45		1.016		1.11		4.393	
0.46		1.050		1.12		4.461	
0.47		1.085		1.13		4.530	
0.48		1.122		1.14		4.600	
0.49		1.161		1.15		4.670	
0.50	1.190	1.200	0.84	1.16		4.740	
0.51		1.239		1.17		4.811	
0.52		1.278		1.18		4.883	
0.53		1.317		1.19		4.955	
0.54		1.356		1.20	4.426	5.027	13.58
0.55		1.396		1.21		5.100	
0.56		1.437		1.22		5.173	
0.57		1.478		1.23		5.247	
0.58		1.519		1.24		5.321	
0.59		1.561		1.25		5.395	
0.60	1.565	1.603	2.43	1.26		5.470	
0.61		1.646		1.27		5.547	
0.62		1.689		1.28		5.624	
0.63		1.732		1.29		5.702	
0.64		1.778		1.30	4.990	5.781	15.85
0.65		1.824		1.31		5.861	

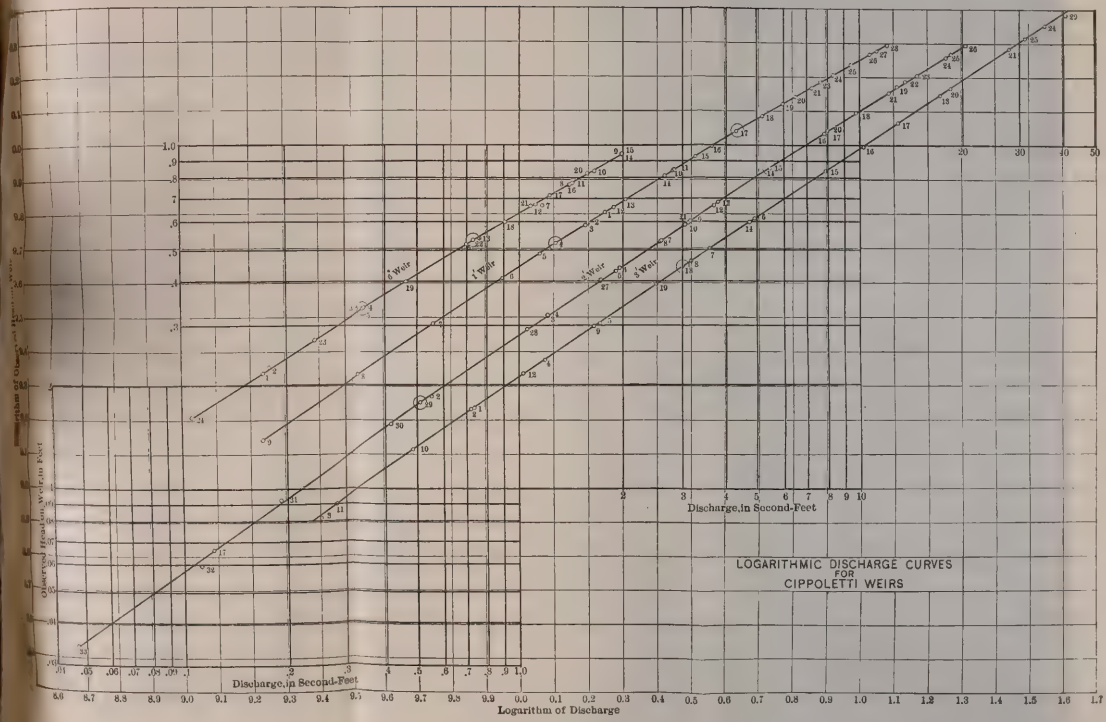
TABLE 3. — (*Continued.*)

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
1.32		5.941		1.67		8.994	
1.33		6.021		1.68		9.088	
1.34		6.102		1.69		9.182	
1.35		6.183		1.70	7.462	9.276	24.31
1.36		6.265		1.71		9.370	
1.37		6.347		1.72		9.464	
1.38		6.429		1.73		9.558	
1.39		6.512		1.74		9.652	
1.40	5.577	6.595	18.25	1.75		9.746	
1.41		6.678		1.76		9.841	
1.42		6.762		1.77		9.937	
1.43		6.846		1.78		10.034	
1.44		6.930		1.79		10.131	
1.45		7.015		1.80	8.130	10.229	25.82
1.46		7.101		1.81		10.327	
1.47		7.187		1.82		10.425	
1.48		7.274		1.83		10.523	
1.49		7.362		1.84		10.624	
1.50	6.185	7.450	20.45	1.85		10.728	
1.51		7.538		1.86		10.833	
1.52		7.626		1.87		10.938	
1.53		7.714		1.88		11.044	
1.54		7.802		1.89		11.150	
1.55		7.890		1.90	8.817	11.256	27.66
1.56		7.979		1.91		11.363	
1.57		8.069		1.92		11.470	
1.58		8.160		1.93		11.577	
1.59		8.251		1.94		11.684	
1.60	6.814	8.343	22.44	1.95		11.791	
1.61		8.435		1.96		11.900	
1.62		8.527		1.97		12.010	
1.63		8.620		1.98		12.120	
1.64		8.713		1.99		12.232	
1.65		8.806		2.00	9.522	12.345	29.65
1.66		8.900					

diate heads, and the entire tank for the high heads. The discharge curves are shown on Plates XXXV and XXXVI, and Tables 4 and 5 give the details.

Plates XXXIV and XXXV show the discharge curves for the actual discharge of the Cippoletti weirs and the theoretical discharge by the Cippoletti formula. It is observed that in every case the actual discharge curve follows the Cippoletti curve to a gauge height of approximately one-third of the length of the crest. After this point, the actual discharges for the gauge height become larger than indicated by the formula. Tables 2 to 5 give the actual discharge for each $\frac{1}{100}$ ft. of gauge height, the formula discharge for each $\frac{1}{10}$ ft. of gauge height, and the percentage that the actual discharge is in excess of the formula discharge for each $\frac{1}{10}$ ft. of gauge height. At a gauge height of 1 ft. on a 6-in. weir, the difference is 30.36%, at 2 ft. on a 1-ft. weir it is 29.65%, showing that at a gauge height equal to twice the length

105A.



LOGARITHMIC DISCHARGE CURVES
FOR
CIPPOLETTI WEIRS

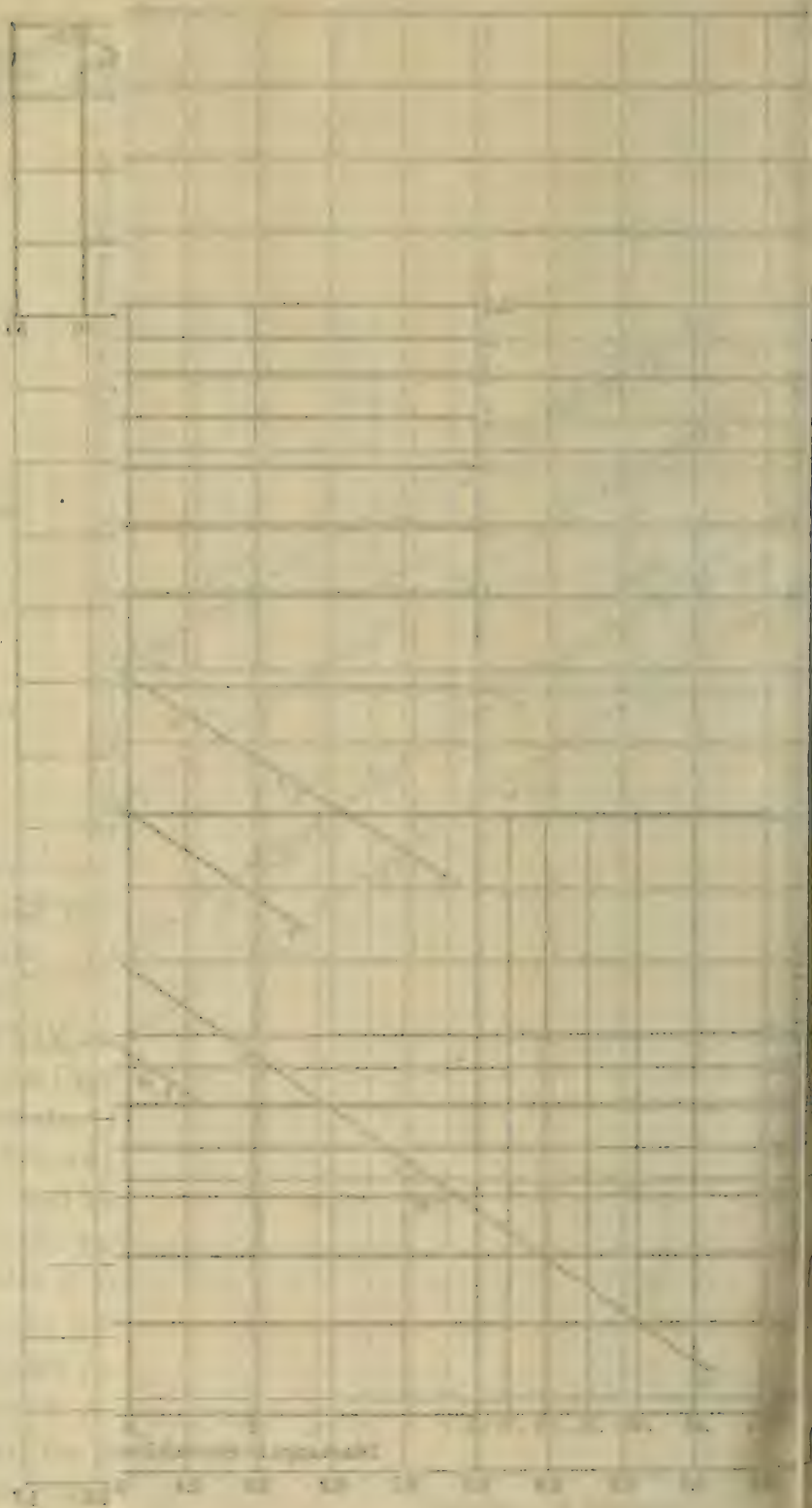


TABLE 4.—GAUGE HEIGHT AND DISCHARGE FOR 2-FOOT CIPPOLETTI WEIR.

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
0.00		0.000		0.66		3.635	
0.01		0.01		0.67		3.725	
0.02		0.02		0.68		3.810	
0.03		0.04		0.69		3.895	
0.04		0.05		0.70	3.94	3.98	1.02
0.05	0.08	0.08		0.71		4.06	
0.06		0.10		0.72		4.15	
0.07		0.12		0.73		4.24	
0.08		0.15		0.74		4.33	
0.09		0.18		0.75		4.415	
0.10	0.21	0.21		0.76		4.51	
0.11		0.25		0.77		4.60	
0.12		0.28		0.78		4.69	
0.13		0.32		0.79		4.78	
0.14		0.35		0.80	4.82	4.87	1.04
0.15		0.39		0.81		4.96	
0.16		0.43		0.82		5.05	
0.17		0.47		0.83		5.14	
0.18		0.51		0.84		5.24	
0.19		0.56		0.85		5.34	
0.20	0.60	0.60		0.86		5.44	
0.21		0.65		0.87		5.54	
0.22		0.69		0.88		5.64	
0.23		0.74		0.89		5.74	
0.24		0.79		0.90	5.75	5.84	1.57
0.25		0.84		0.91		5.94	
0.26		0.89		0.92		6.04	
0.27		0.94		0.93		6.14	
0.28		1.00		0.94		6.25	
0.29		1.05		0.95		6.36	
0.30	1.11	1.11		0.96		6.47	
0.31		1.16		0.97		6.68	
0.32		1.22		0.98		6.69	
0.33		1.28		0.99		6.80	
0.34		1.33		1.00	6.73	6.91	2.67
0.35		1.39		1.01		7.02	
0.36		1.45		1.02		7.13	
0.37		1.52		1.03		7.24	
0.38		1.58		1.04		7.35	
0.39		1.64		1.05		7.47	
0.40	1.70	1.70		1.06		7.59	
0.41		1.77		1.07		7.71	
0.42		1.83		1.08		7.83	
0.43		1.90		1.09		7.95	
0.44		1.97		1.10	7.77	8.07	3.86
0.45	2.03	2.03		1.11		8.19	
0.46		2.10		1.12		8.31	
0.47		2.17		1.13		8.43	
0.48		2.24		1.14		8.55	
0.49		2.31		1.15		8.67	
0.50	2.38	2.38		1.16		8.79	
0.51		2.45		1.17		8.91	
0.52		2.53		1.18		9.03	
0.53		2.605		1.19		9.15	
0.54		2.68		1.20	8.85	9.28	4.86
0.55	2.75	2.755	0.18	1.21		9.40	
0.56		2.83		1.22		9.52	
0.57		2.905		1.23		9.65	
0.58		2.98		1.24		9.78	
0.59		3.06		1.25		9.91	
0.60	3.13	3.14	0.32	1.26		10.04	
0.61		3.22		1.27		10.17	
0.62		3.30		1.28		10.30	
0.63		3.38		1.29		10.43	
0.64		3.465		1.30	9.98	10.56	5.81
0.65		3.55		1.31		10.69	

TABLE 4.—(*Continued.*)

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
1.32		10.82		1.67		15.86	
1.33		10.95		1.68		16.02	
1.34		11.08		1.69		16.18	
1.35		11.21		1.70	14.92	16.34	9.52
1.36		11.35		1.71		16.50	
1.37		11.49		1.72		16.66	
1.38		11.63		1.73		16.82	
1.39		11.77		1.74		16.98	
1.40	11.15	11.91	6.82	1.75		17.14	
1.41		12.05		1.76		17.30	
1.42		12.19		1.77		17.46	
1.43		12.33		1.78		17.62	
1.44		12.47		1.79		17.78	
1.45		12.61		1.80	16.26	17.94	10.33
1.46		12.75		1.81		18.10	
1.47		12.89		1.82		18.26	
1.48		13.03		1.83		18.42	
1.49		13.17		1.84		18.58	
1.50	12.37	13.31	7.60	1.85		18.74	
1.51		13.45		1.86		18.91	
1.52		13.59		1.87		19.08	
1.53		13.73		1.88		19.25	
1.54		13.88		1.89		19.42	
1.55		14.03		1.90	17.63	19.59	11.12
1.56		14.18		1.91		19.76	
1.57		14.33		1.92		19.93	
1.58		14.48		1.93		20.10	
1.59		14.63		1.94		20.27	
1.60	13.63	14.78	8.44	1.95		20.45	
1.61		14.93		1.96		20.63	
1.62		15.08		1.97		20.81	
1.63		15.23		1.98		20.99	
1.64		15.38		1.99		21.17	
1.65		15.54		2.00	19.04	21.35	12.13
1.66		15.70					

of the crest the actual discharge is approximately 30% in excess of the formula discharge. The gauge heights were not carried up to this proportion on the 2-ft. and 3-ft. weirs, but it will be observed that the percentage of difference for the same proportion of gauge height to length of crest agrees fairly well for the different weirs.

Plate XXXVI shows the logarithmic discharge curves for each Cippoletti weir. For the 6-in., 1-ft., and 2-ft. weirs, it is observed that there are three distinct straight lines for each weir. For each succeeding larger weir, the points plot in less decided breaks, and for the 3-ft. weir, there are only two curves. The equation of each of these separate divisions was taken, and the gauge height at which the change in equation takes place was observed, also the proportional part that each of these gauge heights is of their respective lengths of crest. These are given in Table 7. It will be observed that, for the 6-in. and 1-ft. weirs these breaks occur at gauge heights which are approxi-

TABLE 5.—GAUGE HEIGHT AND DISCHARGE FOR 3-FOOT CIPPOLETTI WEIR.

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
0.00		0.00		0.66		5.51	
0.01		0.01		0.67		5.64	
0.02		0.02		0.68		5.77	
0.03		0.05		0.69		5.90	
0.04		0.08		0.70	5.92	6.03	1.86
0.05	0.11	0.12		0.71		6.16	
0.06		0.16		0.72		6.29	
0.07		0.20		0.73		6.42	
0.08		0.24		0.74		6.55	
0.09		0.28		0.75	6.56	6.68	
0.10	0.32	0.33		0.76		6.81	
0.11		0.38		0.77		6.94	
0.12		0.43		0.78		7.08	
0.13		0.48		0.79		7.22	
0.14		0.53		0.80	7.23	7.36	1.80
0.15	0.59	0.59		0.81		7.50	
0.16		0.65		0.82		7.64	
0.17		0.71		0.83		7.78	
0.18		0.77		0.84		7.92	
0.19		0.84		0.85	7.92	8.06	
0.20	0.90	0.91		0.86		8.20	
0.21		0.98		0.87		8.34	
0.22		1.05		0.88		8.48	
0.23		1.12		0.89		8.62	
0.24		1.19		0.90	8.62	8.77	1.74
0.25	1.26	1.27		0.91		8.92	
0.26		1.35		0.92		9.07	
0.27		1.43		0.93		9.22	
0.28		1.51		0.94		9.37	
0.29		1.59		0.95		9.52	
0.30	1.66	1.67		0.96		9.67	
0.31		1.75		0.97		9.82	
0.32		1.84		0.98		9.98	
0.33		1.93		0.99		10.14	
0.34		2.02		1.00	10.10	10.30	1.98
0.35	2.09	2.11		1.01		10.46	
0.36		2.20		1.02		10.62	
0.37		2.29		1.03		10.78	
0.38		2.38		1.04		10.94	
0.39		2.47		1.05		11.10	
0.40	2.56	2.57		1.06		11.26	
0.41		2.67		1.07		11.42	
0.42		2.77		1.08		11.58	
0.43		2.87		1.09		11.74	
0.44		2.97		1.10	11.65	11.90	2.15
0.45	3.05	3.07	0.66	1.11		12.06	
0.46		3.17		1.12		12.22	
0.47		3.27		1.13		12.38	
0.48		3.38		1.14		12.54	
0.49		3.49		1.15		12.71	
0.50	3.57	3.60	0.84	1.16		12.88	
0.51		3.71		1.17		13.05	
0.52		3.82		1.18		13.22	
0.53		3.93		1.19		13.39	
0.54		4.04		1.20	13.28	13.56	2.11
0.55	4.12	4.16		1.21		13.73	
0.56		4.28		1.22		13.91	
0.57		4.40		1.23		14.09	
0.58		4.52		1.24		14.27	
0.59		4.64		1.25		14.45	
0.60	4.69	4.76	1.49	1.26		14.63	
0.61		4.88		1.27		14.81	
0.62		5.00		1.28		14.99	
0.63		5.12		1.29		15.17	
0.64		5.25		1.30	14.97	15.35	2.54
0.65	5.29	5.38		1.31		15.53	

TABLE 5.—(*Continued.*)

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
1.32		15.71		1.89		27.35	
1.33		15.89		1.90	26.45	27.57	4.23
1.34		16.07		1.91		27.79	
1.35	15.84	16.25		1.92		28.01	
1.36		16.44		1.93		28.23	
1.37		16.63		1.94		28.45	
1.38		16.82		1.95		28.67	
1.39		17.01		1.96		28.90	
1.40	16.73	17.20	2.81	1.97		29.13	
1.41		17.39		1.98		29.36	
1.42		17.58		1.99		29.59	
1.43		17.77		2.00	28.57	29.82	4.38
1.44		17.96		2.01		30.05	
1.45		18.15		2.02		30.28	
1.46		18.34		2.03		30.51	
1.47		18.53		2.04		30.74	
1.48		18.72		2.05		30.97	
1.49		18.91		2.06		31.20	
1.50	18.56	19.10	2.91	2.07		31.43	
1.51		19.30		2.08		31.66	
1.52		19.50		2.09		31.89	
1.53		19.70		2.10	30.74	32.13	4.52
1.54		19.90		2.11		32.37	
1.55		20.10		2.12		32.61	
1.56		20.30		2.13		32.85	
1.57		20.50		2.14		33.09	
1.58		20.70		2.15		33.33	
1.59		20.90		2.16		33.57	
1.60	20.44	21.11	3.27	2.17		33.81	
1.61		21.32		2.18		34.05	
1.62		21.53		2.19		34.29	
1.63		21.74		2.20	32.96	34.53	4.76
1.64		21.95		2.21		34.77	
1.65		22.16		2.22		35.01	
1.66		22.37		2.23		35.25	
1.67		22.58		2.24		35.49	
1.68		22.79		2.25		35.73	
1.69		23.00		2.26		35.97	
1.70	22.39	23.21	3.66	2.27		36.21	
1.71		23.42		2.28		36.45	
1.72		23.63		2.29		36.70	
1.73		23.84		2.30	35.23	36.95	4.88
1.74		24.05		2.31		37.20	
1.75		24.27		2.32		37.45	
1.76		24.49		2.33		37.70	
1.77		24.71		2.34		37.95	
1.78		24.93		2.35		38.20	
1.79		25.15		2.36		38.45	
1.80	24.39	25.37	4.02	2.37		38.70	
1.81		25.59		2.38		38.95	
1.82		25.81		2.39		39.20	
1.83		26.03		2.40	37.55	39.45	5.06
1.84		26.25		2.41		39.70	
1.85		26.47		2.42		39.95	
1.86		26.63		2.43		40.20	
1.87		26.91		2.44		40.45	
1.88		27.13		2.45		40.70	

mately one-half the length of the crest and equal to the length of crest, respectively.

For the 2-ft. weir, the heads are not high enough to obtain the break at the length of crest, but the break at about one-half that length is quite evident. This weir makes a change in equation at a lower

proportion than the other two, at approximately one-tenth of the length.

For the 3-ft. weir, there are only two equations, the break occurring at a proportion of gauge height to length of crest of about one-tenth.

Table 1 gives the results of the experiments as recorded, and also, in each run, the value of C in the formula, $Q = Clh^{\frac{3}{2}}$, the customary weir formula.

Table 6 gives the gauge heights of the experiments, arranged in the order of their value, and the corresponding values of C. This table shows that the values of C increase with the gauge height, and that the exponent, $\frac{3}{2}$, or the slope of the logarithmic curve, is not correct, as is also shown by the logarithmic curves of actual discharge.

Table 8 gives the gauge heights for every $\frac{5}{100}$ ft. up to a height of 2.45 ft. These gauge heights are accompanied by the corresponding discharges for each of the four Cippoletti weirs tested. From this table it is seen that the discharges by formula are practically correct up to a gauge height of one-third of the length of the crest. Above this point, however, the use of a new formula or discharge table is essential to accuracy. This table also shows that, above the limits of the formula, the weirs do not discharge in proportion to their length of crest, and that this difference increases with the gauge height. It will be observed that the discharges of the 2-ft. and 3-ft. weirs are more in proportion to the length of the crest than any of the others.

Table 9 shows the values of C in the weir formula, $Q = Clh^{\frac{3}{2}}$ for all the Cippoletti weirs. The values were prepared for each $\frac{5}{100}$ ft. of gauge height, up to a head of 0.3 ft., and for every $\frac{1}{10}$ ft. above that point. The values of Q in the formula were taken from the prepared discharge tables. This table shows that above a gauge height equal to one-third of the length of the crest, C increases very rapidly in each case, and that this increase is not uniform for all the weirs, but is more nearly in proportion to the ratio of gauge height to length of crest.

The foregoing tables and plates show that the Cippoletti weir formula may be used with reasonable accuracy for depths up to one-third of the length of the crest. Above this gauge height, it will be necessary to use tables prepared carefully from actual ratings. The

TABLE 6.—Gauge Height and Values of C in the Formula, $Q = Clh^{\frac{3}{2}}$

Cippoletti Weirs.

Series No.	Run No.	Average gauge height.	C.
	6-Inch Weir.		
1	24	0.161	3.287
	1	0.217	3.362
	2	0.225	3.164
	23	0.270	3.478
	3	0.334	3.304
	5	0.334	3.552
	4	0.336	3.562
	19	0.400	3.676
	6	0.513	3.783
	22	0.530	3.795
	13	0.537	3.867
	18	0.600	3.898
	7	0.667	4.284
	21	0.667	3.954
	12	0.675	4.028
	17	0.715	4.055
	8	0.770	4.132
	16	0.774	4.126
	11	0.776	4.166
	20	0.830	4.181
	10	0.839	4.291
	14	0.940	4.338
	9	0.941	4.329
	15	0.945	4.337
	1-Foot Weir.		
7	9	0.138	3.333
	8	0.215	3.320
	7	0.302	3.361
	6	0.409	3.394
	5	0.482	3.422
	4	0.518	3.404
	3	0.588	3.460
	2	0.594	3.454
	1	0.637	3.487
	12	0.659	3.520
	13	0.693	3.530
	14	0.815	3.595
	10	0.840	3.599
	11	0.846	3.604
	15	0.927	3.656
	16	0.992	3.692
	17	1.097	3.758
	18	1.219	3.835
	19	1.320	3.913
	20	1.386	3.971
	21	1.476	4.034
	22	.....	.....
	23	1.528	4.076
	24	1.604	4.121
	25	1.721	4.186
	26	1.853	4.265
	27	1.897	4.312
	28	1.978	4.366
	2-Foot Weir.		
8	33	0.034	3.730
	32	0.059	3.889

Series No.	Run No.	Average gauge height.	C.
	2-Foot Weir.—(*Continued.*)		
8	1	0.066	3.588
	31	0.092	3.459
	30	0.154	3.455
	29	0.178	3.409
	2	0.186	3.441
	28	0.290	3.374
	3	0.317	3.392
	4	0.318	3.380
	27	0.406	3.338
	5	0.430	3.386
	6	0.438	3.368
	8	0.524	3.385
	7	0.526	3.385
	11	0.588	3.371
	10	0.594	3.384
	9	0.601	3.380
	12	0.668	3.411
	13	0.679	3.197
	14	0.853	3.445
	15	0.854	3.448
	16	1.086	3.499
	17	1.090	3.514
	20	1.101	3.494
	18	1.238	3.557
	21	1.425	3.601
	19	1.484	3.628
	22	1.537	3.607
	23	1.598	3.689
	24	1.816	3.719
	25	1.852	3.722
	26	1.987	3.737
	3-Foot Weir.		
11	3	0.082	3.514
	11	0.090	3.469
	10	0.130	3.461
	2	0.170	3.433
	1	0.172	3.436
	12	0.216	3.410
	4	0.236	3.450
	9	0.297	3.393
	5	0.302	3.430
	19	0.396	3.357
	13	0.444	3.366
	8	0.460	3.386
	7	0.500	3.400
	14	0.598	3.400
	6	0.609	3.414
	15	0.844	3.413
	16	0.994	3.445
	17	1.167	3.435
	18	1.410	3.463
	20	1.470	3.489
	21	1.923	3.502
	22	.....	.....
	25	2.065	3.526
	24	2.246	3.532
	23	2.415	3.676

TABLE 7.—Equations and Location of Breaks in Logarithmic Curves.

Cippoletti Weirs.

Equation.	Gauge height of break.	Gauge height of break divided by length of weir crest.
6-Inch Weir.		
$4.000\ lh^{1.597}$	.....	.....
$4.178\ lh^{1.633}$	0.327	0.654
$4.409\ lh^{1.71}$	0.530	1.060
1-Foot Weir.		
$3.467\ lh^{1.515}$	.....	.....
$3.690\ lh^{1.61}$	0.518	0.518
$3.631\ lh^{1.75}$	1.109	1.109
2-Foot Weir.		
$3.083\ lh^{1.44}$	.....	.....
$3.388\ lh^{1.50}$	0.178	0.089
$3.443\ lh^{1.63}$	0.864	0.432
3-Foot Weir.		
$3.333\ lh^{1.49}$	.....	.....
$3.435\ lh^{1.526}$	0.444	0.148

results show that the discharge curves of smaller weirs above a gauge height of one-third of the length of the crest are not a single section of a parabola, but a series of short sections of parabolas. This series of parabolas, however, seems to become less evident as the weirs increase in length, and it will be assumed that the logarithmic curve for the 3-ft. weir above a gauge height of 0.444 ft. is a good representative of the behavior of the larger weirs, and that it may be used with reasonable accuracy up to a depth equal to the length of the weir crest. Thus, in the use of Cippoletti weirs, the formula will be taken as correct up to gauge heights of one-third of the length of the crest. Above this point, for 3-ft. weirs and smaller, the tables given here-

with are recommended, and for weirs of greater length than 3 ft., for gauge heights of more than one-third of the length of the crest, the formula, $3.435\ lh^{1.526}$, should be used. In this manner accurate results are assured for all reasonable gauge heights.

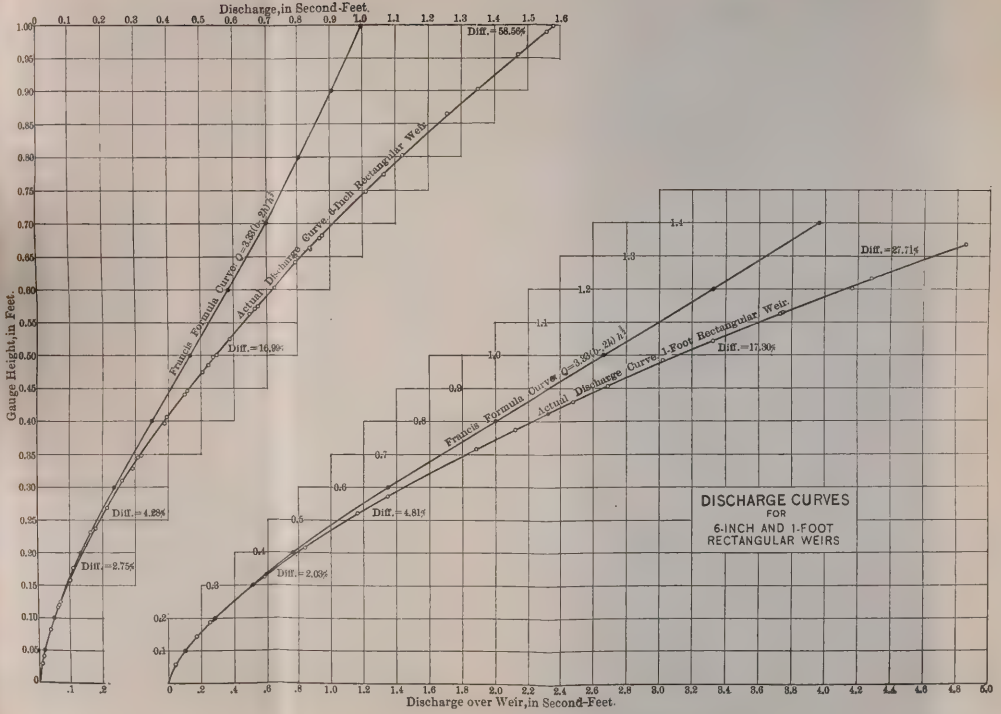
Rectangular Weirs.—The experiments on sharp-crested rectangular weirs were made on the same sizes as were those of the Cippoletti type, namely, 6-in., 1-ft., 2-ft., and 3-ft. These were tested and calibrated at the same headings and in the same manner as the

TABLE 8.—Gauge Heights and Discharge for Cippoletti Weirs.

Gauge height, in feet.	Discharge of 6-in. weir.	Discharge of 1-ft. weir.	Discharge of 2-ft. weir.	Discharge of 3-ft. weir.
0.00	0.000	0.000	0.000	0.00
0.05	0.019	0.038	0.08	0.12
0.10	0.053	0.107	0.21	0.33
0.15	0.098	0.196	0.39	0.59
0.20	0.151	0.301	0.60	0.91
0.25	0.214	0.421	0.84	1.27
0.30	0.288	0.553	1.11	1.67
0.35	0.371	0.697	1.39	2.11
0.40	0.463	0.852	1.70	2.57
0.45	0.563	1.016	2.03	3.07
0.50	0.669	1.200	2.38	3.60
0.55	0.781	1.396	2.755	4.16
0.60	0.906	1.603	3.14	4.76
0.65	1.043	1.824	3.55	5.38
0.70	1.186	2.066	3.98	6.03
0.75	1.336	2.312	4.415	6.68
0.80	1.492	2.565	4.87	7.36
0.85	1.657	2.830	5.34	8.06
0.90	1.832	3.105	5.84	8.77
0.95	2.012	3.389	6.36	9.52
1.00	2.194	3.686	6.91	10.30
1.05	.....	3.999	7.47	11.10
1.10	.....	4.325	8.07	11.90
1.15	.....	4.670	8.67	12.71
1.20	.....	5.027	9.28	13.56
1.25	.....	5.395	9.91	14.45
1.30	.....	5.781	10.56	15.35
1.35	.....	6.183	11.21	16.25
1.40	.....	6.595	11.91	17.20
1.45	.....	7.015	12.61	18.15
1.50	.....	7.450	13.31	19.10
1.55	.....	7.890	14.03	20.10
1.60	.....	8.343	14.78	21.11
1.65	.....	8.806	15.54	22.16
1.70	.....	9.276	16.34	23.21
1.75	.....	9.746	17.14	24.27
1.80	.....	10.229	17.94	25.37
1.85	.....	10.728	18.74	26.47
1.90	.....	11.256	19.59	27 57
1.95	.....	11.791	20.45	28.67
2.00	.....	12.345	21.35	29.82
2.05	.....	.....	.....	30.97
2.10	.....	.....	.....	32.13
2.15	.....	.....	.....	33.33
2.20	.....	.....	.....	34.53
2.25	.....	.....	.....	35.73
2.30	.....	.....	.....	36.95
2.35	.....	.....	.....	38.20
2.40	.....	.....	.....	39.45
2.45	.....	.....	.....	40.70

PLATE XXXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1264.
STEWARD AND LONGWELL ON
WEIR DISCHARGE EXPERIMENTS.

1064A.



trapezoidal weirs. No velocity of approach existed in any case. Full contraction and a perfect, undisturbed, and free nappe were maintained in every run. The different compartments of the measuring tank were used as the heads and discharge varied.

TABLE 9.—Gauge Height and Values of C in the Formula, $Q = Clh^{\frac{3}{2}}$, for the Cippoletti Weir.

Gauge height.	Length of weir, in feet.			
	6-in.	1-ft.	2-ft.	3-ft.
0.0				
0.05	3.367	3.367	3.367	3.571
0.10	3.367	3.367	3.367	3.481
0.15	3.367	3.367	3.367	3.367
0.20	3.367	3.367	3.367	3.393
0.25	3.424	3.367	3.367	3.387
0.30	3.506	3.367	3.367	3.388
0.40	3.660	3.367	3.367	3.386
0.50	3.784	3.394	3.367	3.394
0.60	3.898	3.449	3.377	3.413
0.70	4.049	3.527	3.398	3.431
0.80	4.170	3.585	3.408	3.428
0.90	4.291	3.637	3.420	3.424
1.00	4.388	3.686	3.455	3.433
1.10	.....	3.749	3.497	3.438
1.20	.....	3.824	3.530	3.438
1.30	.....	3.900	3.562	3.452
1.40	.....	3.981	3.595	3.461
1.50	.....	4.053	3.621	3.464
1.60	.....	4.122	3.651	3.477
1.70	.....	4.186	3.687	3.491
1.80	.....	4.235	3.714	3.501
1.90	.....	4.298	3.740	3.509
2.00	.....	4.365	3.775	3.515
2.10	.....	.....	.....	3.519
2.20	.....	.....	.....	3.527
2.30	.....	.....	.....	3.531
2.40	.....	.....	.....	3.537

Tables and plates similar to those for the Cippoletti weirs were prepared for this type. The Francis formula for rectangular weirs was used, and in each case the percentage of difference between the actual and Francis discharges was tabulated.

Table 10 gives the actual experimental data for each weir, together with the corresponding values of C in the formula, $Q = Clh^{\frac{3}{2}}$.

Tables 11 to 14 give the actual discharges corresponding to each $\frac{1}{100}$ ft. of gauge height, up to the limit of the experiments. The discharges, computed from the Francis formula, $Q = 3.333\ (l - 0.2h)\ h^{\frac{3}{2}}$, are tabulated for each $\frac{1}{10}$ ft. of gauge height, and the percentage of difference between this and the actual discharge is recorded as shown. It is seen that the formula is only fairly correct for very low heads.

TABLE 10.—Results of Experiments on Weirs.
Rectangular Weirs.

Series No.	Run No.	Average gauge height, in feet.	Variation in gauge height.	Length of weir, in inches.	Time interval, in seconds.	Actual discharge, in cubic feet.	Discharge, in second-feet.	C in formula, $Q = Clh^{\frac{3}{2}}$
		h		l			Q	
4	1	0.117	0.018	6	1 200	77	0.064	3.192
	2	0.112	0.000	6	1 200	75	0.062	3.307
	3	0.156	0.012	6	1 200	120	0.100	3.247
	4	0.236	0.006	6	900	162	0.180	3.139
	5	0.348	0.004	6	600	194	0.323	3.147
	6	0.499	0.006	6	600	331	0.552	3.132
	7	0.170	0.006	6	1 200	129	0.108	3.081
	8	0.082	0.006	6	2 400	93	0.039	3.319
	9	0.042	0.000	6	3 600	52	0.014	3.256
	10	0.030	0.006	6	3 600	31	0.009	3.462
	11	0.678	0.019	6	600	524	0.873	3.128
	12	0.574	0.006	6	600	406	0.677	3.113
	13	0.774	0.005	6	600	642	1.070	3.142
	14	0.866	0.015	6	600	757	1.262	3.132
	15	0.992	0.019	6	540	845	1.565	3.168
	16	0.751	0.000	6	600	612	1.020	3.135
	17	0.803	0.006	6	600	678	1.130	3.141
	18	0.902	0.001	6	600	813	1.355	3.163
	19	0.955	0.004	6	600	884	1.473	3.157
	20	0.999	0.018	6	600	949	1.582	3.169
	21	0.682	0.030	6	600	528	0.880	3.125
	22	0.572	0.002	6	420	282	0.671	3.102
	23	0.564	0.007	6	600	392	0.653	3.083
	24	0.487	0.006	6	600	317	0.528	3.108
	25	0.446	0.009	6	600	276	0.460	3.088
	26	0.405	0.006	6	600	239	0.398	3.088
	27	0.346	0.003	6	600	187	0.312	3.065
	28	0.309	0.004	6	600	157	0.262	3.050
	29	0.231	0.019	6	600	100	0.167	3.009
	30	0.110	0.010	6	2 400	143	0.060	3.288
	31	0.154	0.006	6	1 800	173	0.096	3.174
	32	0.208	0.004	6	900	134	0.149	3.143
	33	0.604	0.005	6	600	439	0.732	3.119
	34	0.524	0.007	6	600	356	0.593	3.127
	35	0.661	0.013	6	600	507	0.845	3.145
	36	0.472	0.006	6	600	303	0.505	3.114
	37	0.662	0.012	6	600	506	0.843	3.130
	38	0.638	0.012	6	600	479	0.798	3.132
	39	0.498	0.007	6	600	326	0.543	3.090
	40	0.442	0.003	6	600	274	0.457	3.110
	41	0.398	0.010	6	600	234	0.390	3.106
	42	0.328	0.011	6	600	176	0.293	3.119
	43	0.268	0.006	6	900	194	0.216	3.112
13	1	1.338	0.009	12	600	2 922	4.870	3.147
	2	1.126	0.018	12	300	1 123	3.743	3.133
	3	1.205	0.004	12	600	2 506	4.177	3.158
	4	1.130	0.009	12	600	2 254	3.757	3.128
	5	1.044	0.010	12	600	1 999	3.332	3.124
	6	0.982	0.013	12	600	1 815	3.025	3.108
	7	0.909	0.002	12	600	1 617	2.695	3.109
	8	0.857	0.005	12	600	1 490	2.483	3.130
	9	0.775	0.008	12	600	1 278	2.130	3.122
	10	0.571	0.012	12	600	814	1.357	3.146
	11	0.714	0.005	12	600	1 131	1.885	3.124
	12	0.518	0.009	12	600	702	1.170	3.138
	13	0.416	0.003	12	900	762	0.847	3.157
	14	0.189	0.010	12	1 200	320	0.267	3.248
	15	0.324	0.005	12	1 200	703	0.586	3.178
	16	0.056	0.009	12	1 500	67	0.045	3.383
	17	0.142	0.005	12	1 800	321	0.178	3.327
	18	0.102	0.015	12	1 020	104	0.102	3.129
	19	0.822	0.011	12	600	1 400	2.333	3.131
	20	1.233	0.008	12	480	2 060	4.292	3.135

TABLE 10.—(*Continued.*)

Series No.	Run No.	Average gauge height, in feet.	Variation in gauge height.	Length of weir.	Time interval, in seconds.	Actual discharge, cubic feet.	Discharge, in second-feet.	C in formula, $Q = Clh^{\frac{3}{2}}$
		h		l			Q	
14	1	0.081	0.002	24	1 800	290	0.161	3.500
	2	0.104	0.008	24	1 200	277	0.231	3.438
	3	0.167	0.003	24	900	416	0.462	3.382
	4	0.233	0.005	24	600	449	0.748	3.324
	5	0.294	0.002	24	600	627	1.045	3.278
	6	0.395	0.008	24	600	970	1.617	3.256
	7	0.510	0.020	24	600	1 417	2.362	3.243
	8	0.573	0.003	24	600	1 669	2.782	3.207
	9	0.717	0.011	24	600	2 321	3.868	3.186
	10	0.910	0.003	24	480	2 635	5.490	3.162
	11	1.102	0.004	24	420	3 063	7.293	3.152
	12	.....	.....	..	.....	.....	.....	.....
	13	1.283	0.014	24	300	2 724	9.080	3.124
	14	1.429	0.015	24	300	3 171	10.570	3.094
	15	0.549	0.002	24	900	2 326	2.584	3.176
	16	0.886	0.007	24	900	4 716	5.240	3.141
	17	1.185	0.005	24	600	4 857	8.095	3.138
	18	1.574	0.005	24	480	5 934	12.362	3.130
	19	0.059	0.009	24	2 100	202	0.096	3.331
	20	0.214	0.004	24	600	394	0.657	3.318
15	1	0.482	0.000	36	600	2 001	3.335	3.322
	2	0.468	0.002	36	600	1 921	3.202	3.333
	3	0.426	0.001	36	630	1 754	2.784	3.337
	4	0.385	0.000	36	600	1 448	2.413	3.367
	5	.....	.....	36	.....	.....	.....	.....
	6	0.101	0.000	36	930	316	0.340	3.531
	7	0.310	0.001	36	600	1 028	1.713	3.308
	8	0.198	0.001	36	900	808	0.898	3.398
	9	0.452	0.001	36	600	1 800	3.000	3.291
	10	0.614	0.002	36	390	1 836	4.708	3.262
	11	0.950	0.003	36	480	4 325	9.010	3.244
	12	0.826	0.003	36	600	4 401	7.335	3.257
	13	1.190	0.007	36	510	6 398	12.545	3.221
	14	2.058	0.004	36	360	10 077	27.992	3.160
	15	2.061	0.005	36	300	8 409	28.030	3.157
	16	1.820	0.007	36	360	8 340	23.167	3.145
	17	1.606	0.010	36	300	5 802	19.340	3.167
	18	1.401	0.003	36	360	5 705	15.847	3.185
	19	1.382	0.004	36	600	9 353	15.588	3.198
	20	1.609	0.004	36	360	6 985	19.403	3.169
	21	0.690	0.001	36	600	3 358	5.597	3.255

Table 15 gives the value of C in the formula, $Q = Clh^{\frac{3}{2}}$, for the gauge heights of the various experiments, arranged in the order of their value. It will be observed that these do not show the marked variation characterizing the Cippoletti weirs. Table 16 gives the equations of the different parabolas, the gauge height at which the equations change, and the proportion that these gauge heights are of the lengths of crest. It may be seen that the equations do not have such a wide range as in the previous case, and that the three equations agree more closely with each other as the size of the weir increases, and finally

TABLE 11.—Gauge Height and Discharge for 6-Inch Rectangular Weir.

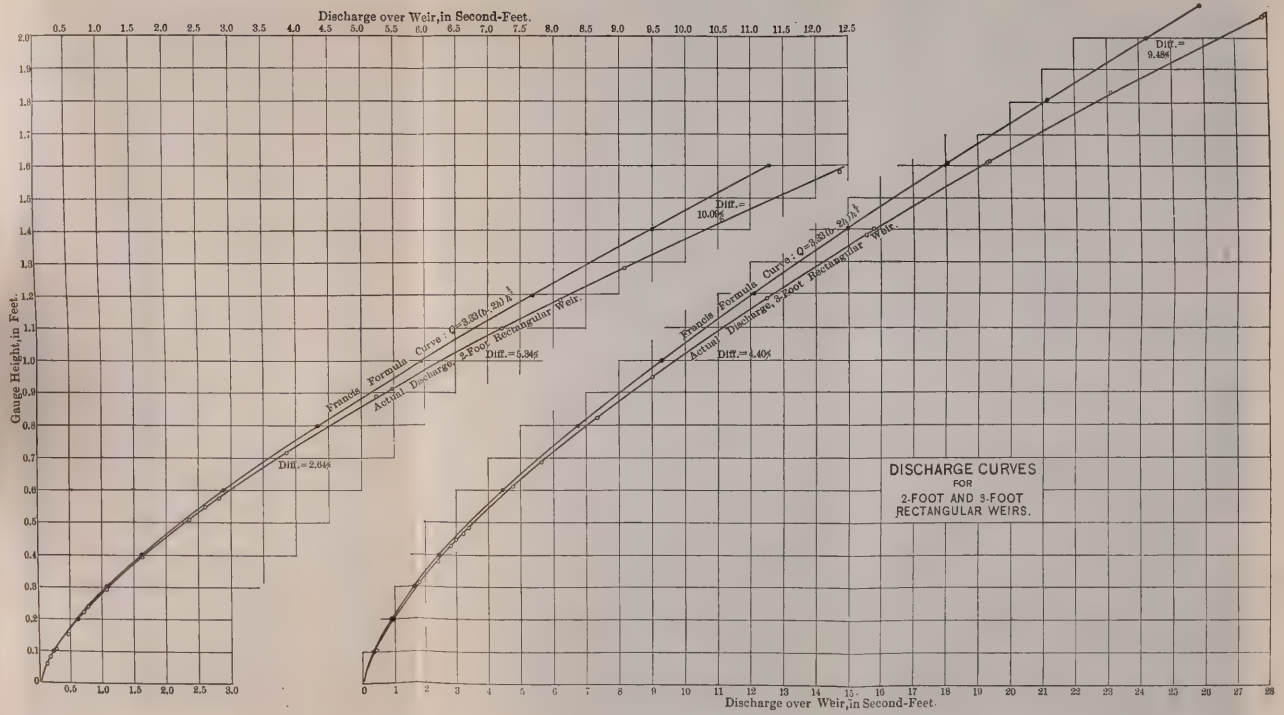
Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.000	0.000		0.51		0.568	
0.01		0.002		0.52		0.585	
0.02		0.005		0.53		0.602	
0.03		0.009		0.54		0.619	
0.04		0.013		0.55		0.636	
0.05	0.018	0.018		0.56		0.653	
0.06		0.024		0.57		0.671	
0.07		0.030		0.58		0.689	
0.08		0.037		0.59		0.707	
0.09		0.044		0.60	0.588	0.725	23.30
0.10	0.050	0.051	2.00	0.61		0.743	
0.11		0.059		0.62		0.761	
0.12		0.067		0.63		0.780	
0.13		0.075		0.64		0.799	
0.14		0.084		0.65		0.818	
0.15		0.093		0.66		0.837	
0.16		0.102		0.67		0.856	
0.17	0.109	0.112	2.75	0.68		0.876	
0.18		0.122		0.69		0.896	
0.19		0.132		0.70	0.702	0.916	30.49
0.20	0.137	0.142	3.65	0.71		0.936	
0.21		0.152		0.72		0.956	
0.22		0.162		0.73		0.976	
0.23		0.173		0.74		0.997	
0.24		0.184		0.75		1.018	
0.25		0.195		0.76		1.039	
0.26		0.206		0.77		1.060	
0.27		0.218		0.78		1.081	
0.28		0.230		0.79		1.103	
0.29		0.242		0.80	0.810	1.125	38.89
0.30	0.241	0.254	5.39	0.81		1.147	
0.31		0.266		0.82		1.169	
0.32		0.279		0.83		1.191	
0.33		0.292		0.84		1.213	
0.34		0.305		0.85		1.235	
0.35		0.318		0.86		1.258	
0.36		0.332		0.87		1.281	
0.37		0.346		0.88		1.304	
0.38		0.361		0.89		1.327	
0.39		0.376		0.90	0.910	1.350	48.35
0.40	0.354	0.391	10.45	0.91		1.373	
0.41		0.406		0.92		1.396	
0.42		0.421		0.93		1.419	
0.43		0.437		0.94		1.442	
0.44		0.453		0.95		1.465	
0.45	0.412	0.469		0.96		1.488	
0.46		0.485		0.97		1.512	
0.47		0.501		0.98		1.536	
0.48		0.517		0.99		1.560	
0.49		0.534		1.00	0.999	1.584	58.56
0.50	0.471	0.551	16.99				

become one parabola in the 3-ft. weir. The breaks occur at about the same relative positions as in the Cippoletti weirs.

Table 17 gives the discharges of each weir for every $\frac{5}{100}$ ft. of gauge height, as far as the experiments were carried. This table shows that these results bear a very marked proportionality to the various lengths of crest.

PLATE XXXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1264.
STEWARD AND LONGWELL ON
WEIR DISCHARGE EXPERIMENTS.

1068A.



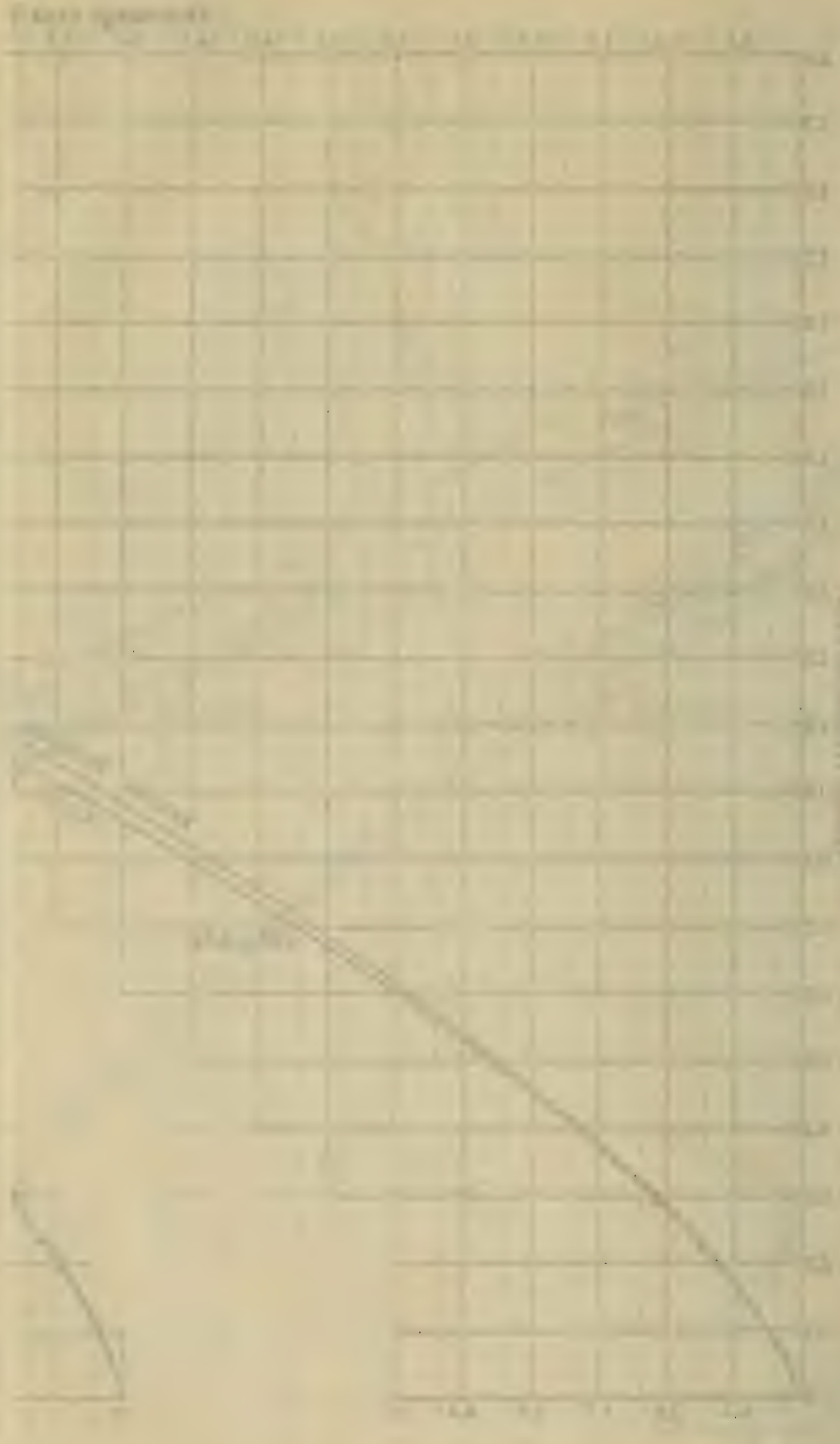


TABLE 12.—Gauge Height and Discharge for 1-Foot Rectangular Weir.

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.000	0.000		0.65		1.640	
0.01		0.005		0.66		1.677	
0.02		0.010		0.67		1.715	
0.03		0.017		0.68		1.753	
0.04		0.026		0.69		1.791	
0.05	0.037	0.037		0.70	1.677	1.830	9.12
0.06		0.048		0.71		1.869	
0.07		0.060		0.72		1.908	
0.08		0.073		0.73		1.948	
0.09		0.087		0.74		1.988	
0.10	0.103	0.103		0.75		2.028	
0.11		0.119		0.76		2.068	
0.12		0.136		0.77		2.108	
0.13		0.153		0.78		2.149	
0.14		0.170		0.79		2.191	
0.15	0.188	0.188		0.80	2.001	2.233	11.5
0.16		0.206		0.81		2.276	
0.17		0.225		0.82		2.319	
0.18		0.245		0.83		2.362	
0.19		0.266		0.84		2.405	
0.20	0.286	0.287	0.35	0.85		2.448	
0.21		0.308		0.86		2.492	
0.22		0.330		0.87		2.536	
0.23		0.352		0.88		2.580	
0.24		0.375		0.89		2.624	
0.25		0.398		0.90	2.331	2.668	14.46
0.26		0.421		0.91		2.712	
0.27		0.445		0.92		2.757	
0.28		0.470		0.93		2.802	
0.29		0.495		0.94		2.847	
0.30	0.514	0.521	1.36	0.95		2.892	
0.31		0.547		0.96		2.938	
0.32		0.574		0.97		2.984	
0.33	0.590	0.602	2.03	0.98		3.031	
0.34		0.630		0.99		3.078	
0.35		0.658		1.00	2.664	3.125	17.30
0.36		0.686		1.01		3.172	
0.37		0.714		1.02		3.219	
0.38		0.743		1.03		3.266	
0.39		0.772		1.04		3.314	
0.40	0.775	0.801	3.35	1.05		3.362	
0.41		0.830		1.06		3.411	
0.42		0.860		1.07		3.460	
0.43		0.890		1.08		3.509	
0.44		0.920		1.09		3.559	
0.45		0.950		1.10	2.997	3.609	20.42
0.46		0.981		1.11		3.659	
0.47		1.012		1.12		3.709	
0.48		1.044		1.13		3.759	
0.49		1.077		1.14		3.809	
0.50	1.060	1.111	4.81	1.15		3.860	
0.51		1.145		1.16		3.911	
0.52		1.179		1.17		3.962	
0.53		1.213		1.18		4.014	
0.54		1.247		1.19		4.066	
0.55		1.282		1.20	3.327	4.119	23.81
0.56		1.317		1.21		4.172	
0.57		1.352		1.22		4.225	
0.58		1.387		1.23		4.278	
0.59		1.423		1.24		4.332	
0.60	1.362	1.459	7.12	1.25		4.387	
0.61		1.495		1.26		4.442	
0.62		1.531		1.27		4.497	
0.63		1.557		1.28		4.552	
0.64		1.603		1.29		4.608	

TABLE 12.—(*Continued.*)

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
1.30	3.652	4.664	27.71	1.36		.....	
1.31		4.720		1.37		.....	
1.32		4.777		1.38		.....	
1.33		4.834		1.39		.....	
1.34		.....		1.40	3.972	.....	
1.35	3.813	.....					

TABLE 13.—GAUGE HEIGHT AND DISCHARGE FOR 2-FOOT RECTANGULAR WEIR.

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.00	0.00		0.51		2.34	
0.01		0.01		0.52		2.41	
0.02		0.025		0.53		2 48	
0.03		0.04		0.54		2.55	
0.04		0.06		0.55		2.62	
0.05	0.08	0.08		0.56		2.69	
0.06		0.10		0.57		2.76	
0.07		0.125		0.58		2.83	
0.08		0.15		0.59		2.90	
0.09		0.18		0.60	2.91	2.98	2.41
0.10	0.21	0.21		0.61		3.05	
0.11		0.24		0.62		3.12	
0.12		0.27		0.63		3.19	
0.13		0.30		0.64		3.26	
0.14		0.34		0.65		3.34	
0.15		0.38		0.66		3.42	
0.16		0.42		0.67	3.41	3.50	2.64
0.17		0.46		0.68		3.58	
0.18		0.50		0.69		3.66	
0.19		0.54		0.70	3.63	3.74	3.08
0.20	0.58	0.58		0.71		3.82	
0.21		0.63		0.72		3.90	
0.22		0.68		0.73		3.98	
0.23		0.73		0.74		4.06	
0.24		0.78		0.75		4.14	
0.25		0.83		0.76		4.22	
0.26		0.88		0.77		4.30	
0.27		0.93		0.78		4.38	
0.28		0.98		0.79		4.46	
0.29		1.03		0.80	4.38	4.54	3.65
0.30	1.06	1.08	1.89	0.81		4.62	
0.31		1.13		0.82		4.70	
0.32		1.18		0.83		4.78	
0.33		1.23		0.84		4.87	
0.34		1.29		0.85		4.96	
0.35		1.35		0.86		5.05	
0.36		1.41		0.87		5.14	
0.37		1.47		0.88		5.23	
0.38		1.53		0.89		5.32	
0.39		1.59		0.90	5.17	5.41	4.64
0.40	1.62	1.65	1.85	0.91		5.50	
0.41		1.71		0.92		5.59	
0.42		1.77		0.93		5.68	
0.43		1.83		0.94		5.77	
0.44		1.89		0.95		5.86	
0.45		1.95		0.96		5.95	
0.46		2.01		0.97		6.04	
0.47		2.07		0.98		6.13	
0.48		2.13		0.99		6.22	
0.49		2.20		1.00	5.99	6.31	5.34
0.50	2.24	2.27	1.34	1.01		6.40	

TABLE 13.—(*Continued.*)

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
1.02		6.49		1.30	8.59	9.27	7.92
1.03		6.58		1.31		9.37	
1.04		6.67		1.32		9.47	
1.05		6.76		1.33		9.58	
1.06		6.85		1.34		9.69	
1.07		6.95		1.35		9.80	
1.08		7.05		1.36		9.91	
1.09		7.15		1.37		10.02	
1.10	6.84	7.25	5.99	1.38		10.13	
1.11		7.35		1.39		10.24	
1.12		7.45		1.40	9.49	10.35	9.06
1.13		7.55		1.41		10.46	
1.14		7.65		1.42		10.57	
1.15		7.75		1.43		10.68	
1.16		7.85		1.44		10.79	
1.17		7.95		1.45		10.90	
1.18		8.05		1.46		11.01	
1.19		8.15		1.47		11.12	
1.20	7.70	8.25	7.14	1.48		11.23	
1.21		8.35		1.49		11.34	
1.22		8.45		1.50	10.40	11.45	10.09
1.23		8.55		1.51		11.56	
1.24		8.65		1.52		11.67	
1.25		8.75		1.53		11.78	
1.26		8.85		1.54		11.89	
1.27		8.95		1.55		12.00	
1.28		9.05		1.56			
1.29		9.15					

Table 18 gives the values of C in the formula, $Q = Clh^{\frac{3}{2}}$, for each of the rectangular weirs, for each $\frac{5}{100}$ ft. of gauge height to 0.3 ft. head, and for each $\frac{1}{10}$ ft. thereafter. These values are based on the discharge, Q, as given in Tables 11 to 14. It will be seen that these values are also variable, both as to gauge height and length of weir crest, thus again indicating the fallacy of the original formula, $Q = Clh^{\frac{3}{2}}$. Table 19 contains the values of C of Table 18 for weirs up to 3 ft. in length and values as given by the late Hamilton Smith, M. Am. Soc. C. E., for the larger weirs. The tables and plates given herewith indicate that the Francis formula is not applicable, within reasonable limits of accuracy, for gauge heights of more than one-fourth of the length of the crest, and also that the errors increase with the size of the weir. The varying values of C indicate, as in the case of the Cippoletti weirs, that the exponent, $\frac{3}{2}$, in the formula, $Q = Clh^{\frac{3}{2}}$, is incorrect, and that if this formula is used it will be necessary to obtain a new value of C for each computation of discharge. Owing to the proportionality existing among the various values of discharge for the same gauge heights, and the tendency toward one equation

TABLE 14.—Gauge Height and Discharge for 3-Foot Rectangular Weir.

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.00	0.00		0.65		5.13	
0.01		0.01		0.66		5.25	
0.02		0.04		0.67		5.37	
0.03		0.07		0.68		5.49	
0.04		0.10		0.69		5.61	
0.05		0.13		0.70	5.58	5.73	2.69
0.06		0.17		0.71		5.85	
0.07		0.21		0.72		5.97	
0.08		0.25		0.73		6.09	
0.09		0.29		0.74		6.22	
0.10	0.31	0.33	6.45	0.75		6.35	
0.11		0.38		0.76		6.48	
0.12		0.43		0.77		6.61	
0.13		0.48		0.78		6.74	
0.14		0.54		0.79		6.87	
0.15		0.60		0.80	6.77	7.00	3.40
0.16		0.66		0.81		7.13	
0.17		0.72		0.82		7.26	
0.18		0.78		0.83		7.39	
0.19		0.84		0.84		7.52	
0.20	0.88	0.91	3.41	0.85		7.65	
0.21		0.98		0.86		7.78	
0.22		1.05		0.87		7.91	
0.23		1.12		0.88		8.05	
0.24		1.19		0.89		8.19	
0.25		1.26		0.90	8.02	8.33	3.87
0.26		1.33		0.91		8.47	
0.27		1.41		0.92		8.61	
0.28		1.49		0.93		8.75	
0.29		1.57		0.94		8.89	
0.30	1.61	1.65	2.48	0.95		9.03	
0.31		1.73		0.96		9.17	
0.32		1.81		0.97		9.31	
0.33		1.90		0.98		9.45	
0.34		1.99		0.99		9.59	
0.35		2.08		1.00	9.32	9.73	4.40
0.36		2.17		1.01		9.87	
0.37		2.26		1.02		10.01	
0.38		2.35		1.03		10.15	
0.39		2.44		1.04		10.30	
0.40	2.46	2.53	2.85	1.05		10.45	
0.41		2.62		1.06		10.60	
0.42		2.71		1.07		10.75	
0.43		2.80		1.08		10.90	
0.44		2.90		1.09		11.05	
0.45		3.00		1.10	10.68	11.20	4.87
0.46		3.10		1.11		11.35	
0.47		3.20		1.12		11.50	
0.48		3.30		1.13		11.65	
0.49		3.40		1.14		11.80	
0.50	3.41	3.50	2.64	1.15		11.95	
0.51		3.60		1.16		12.10	
0.52		3.70		1.17		12.25	
0.53		3.80		1.18		12.40	
0.54		3.90		1.19		12.55	
0.55		4.01		1.20	12.08	12.70	5.13
0.56		4.12		1.21		12.85	
0.57		4.23		1.22		13.00	
0.58		4.34		1.23		13.15	
0.59		4.45		1.24		13.31	
0.60	4.46	4.56	2.24	1.25		13.47	
0.61		4.67		1.26		13.63	
0.62		4.78		1.27		13.79	
0.63		4.89		1.28		13.95	
0.64		5.01		1.29		14.11	

PLATE XXXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1264.
STEWARD AND LONGWELL ON
WEIR DISCHARGE EXPERIMENTS.

1072A.

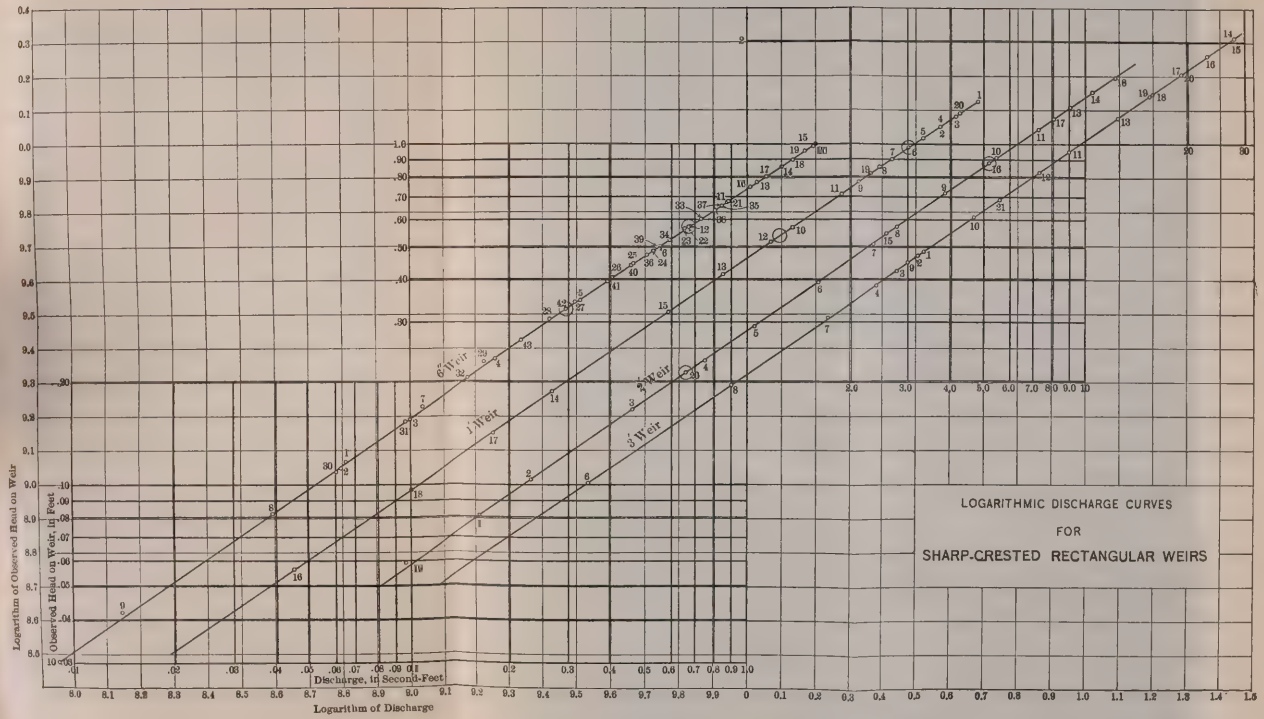


TABLE 14.—(*Continued.*)

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
1.30	13.52	14.27	5.55	1.71		21.19	
1.31		14.43		1.72		21.37	
1.32		14.59		1.73		21.55	
1.33		14.75		1.74		21.73	
1.34		14.91		1.75		21.91	
1.35		15.07		1.76		22.10	
1.36		15.23		1.77		22.29	
1.37		15.39		1.78		22.48	
1.38		15.55		1.79		22.67	
1.39		15.71		1.80	21.23	22.86	7.68
1.40	15.00	15.87	5.8	1.81		23.05	
1.41		16.03		1.82		23.24	
1.42		16.19		1.83		23.43	
1.43		16.35		1.84		23.62	
1.44		16.51		1.85		23.81	
1.45		16.68		1.86		24.00	
1.46		16.85		1.87		24.20	
1.47		17.02		1.88		24.40	
1.48		17.19		1.89		24.60	
1.49		17.36		1.90	22.85	24.80	8.53
1.50	16.53	17.53	6.05	1.91		25.00	
1.51		17.70		1.92		25.20	
1.52		17.87		1.93		25.40	
1.53		18.04		1.94		25.60	
1.54		18.21		1.95		25.80	
1.55		18.38		1.96		26.00	
1.56		18.55		1.97		26.20	
1.57		18.72		1.98		26.40	
1.58		18.89		1.99		26.60	
1.59		19.06		2.00	24.48	26.80	9.48
1.60	18.06	19.23	6.48	2.01		27.00	
1.61		19.40		2.02		27.20	
1.62		19.57		2.03		27.40	
1.63		19.75		2.04		27.60	
1.64		19.93		2.05		27.81	
1.65		20.11		2.06		28.02	
1.66		20.29		2.07		28.23	
1.67		20.47		2.08		28.44	
1.68		20.65		2.09		28.65	
1.69		20.83		2.10	26.14	28.86	10.41
1.70	19.63	21.01	7.03				

in the logarithmic curves, it is recommended that, for weirs more than 3 ft. in length, the equation, $3.242\ lh^{1.46}$ be used; for the smaller weirs, the tables give the desired results.

As an alternative, Table 19 contains the values of C for various gauge heights and sizes of weirs. By means of these values, discharge tables may be prepared for any size of weir up to 19 ft. in length.

In the foregoing experiments, no attempt has been made to develop a formula that will apply to all weirs, but merely to obtain ratings, as far as possible, and a formula or table that may be used with a reasonable degree of accuracy for large weirs with heads higher than previously tested. In this manner sufficiently accurate tables can be made and applied to irrigation work.

TABLE 15.—Gauge Height and Values of C in the Formula, $Q = Clh^{\frac{3}{2}}$

Rectangular Weirs.

Series No.	Run No.	Average gauge height.	C.
	6-Inch Weir.		
4	10	0.030	3.462
	9	0.042	3.256
	8	0.082	3.319
	30	0.110	3.288
	2	0.112	3.307
	1	0.117	3.192
	31	0.154	3.174
	3	0.156	3.247
	7	0.170	3.081
	32	0.208	3.143
	29	0.231	3.009
	4	0.236	3.139
	43	0.268	3.112
	28	0.309	3.050
	42	0.328	3.119
	27	0.346	3.065
	5	0.348	3.147
	41	0.398	3.106
	26	0.405	3.088
	40	0.442	3.110
	25	0.446	3.088
	36	0.472	3.114
	24	0.487	3.108
	39	0.498	3.090
	6	0.499	3.132
	34	0.524	3.127
	23	0.564	3.083
	22	0.572	3.102
	12	0.574	3.113
	33	0.604	3.119
	38	0.638	3.132
	35	0.661	3.145
	37	0.662	3.130
	11	0.678	3.128
	21	0.682	3.125
	16	0.751	3.135
	13	0.774	3.142
	17	0.803	3.141
	14	0.866	3.132
	18	0.902	3.163
	19	0.955	3.157
	15	0.992	3.168
	20	0.999	3.169
	1-Foot Weir.		
13	16	0.056	3.383
	18	0.102	3.129
	17	0.142	3.327
	14	0.189	3.248
	15	0.324	3.178
	13	0.416	3.157
	12	0.518	3.138
	10	0.571	3.146
	11	0.714	3.124
	9	0.775	3.122
	19	0.822	3.131
	1-Foot Weir.—(*Continued.*)		
13	8	0.857	3.130
	7	0.909	3.109
	6	0.982	3.108
	5	1.044	3.124
	2	1.126	3.133
	4	1.130	3.128
	3	1.205	3.158
	20	1.233	3.135
	1	1.338	3.147
	2-Foot Weir.		
14	19	0.059	3.331
	1	0.081	3.500
	2	0.104	3.438
	3	0.167	3.382
	20	0.214	3.318
	4	0.233	3.324
	5	0.294	3.278
	6	0.395	3.256
	7	0.510	3.243
	15	0.549	3.176
	8	0.573	3.207
	9	0.717	3.186
	16	0.886	3.141
	10	0.910	3.162
	11	1.102	3.152
	12	.....	.....
	17	1.185	3.138
	13	1.283	3.124
	14	1.429	3.094
	18	1.574	3.130
	3-Foot Weir.		
15	6	0.101	3.531
	8	0.198	3.398
	7	0.310	3.308
	4	0.385	3.367
	5	.....	.....
	3	0.426	3.337
	9	0.452	3.291
	2	0.468	3.333
	1	0.482	3.322
	10	0.614	3.262
	21	0.690	3.255
	12	0.826	3.257
	11	0.950	3.244
	13	1.190	3.221
	19	1.382	3.198
	18	1.401	3.185
	17	1.606	3.167
	20	1.609	3.169
	16	1.820	3.145
	14	2.058	3.160
	15	2.061	3.157

TABLE 16.—Equations and Location of Breaks in Logarithmic Curves.

Rectangular Weirs.

Equation.	Gauge height of break.	Gauge height of break divided by length of weir crest.
6-Inch Weir.		
$2.958\ lh^{1.45}$	.....	.....
$3.134\ lh^{1.50}$	0.328	0.656
$3.170\ lh^{1.52}$	0.572	1.144
1-Foot Weir.		
$3.084\ lh^{1.47}$	.....	.....
$3.123\ lh^{1.49}$	0.540	0.540
$3.126\ lh^{1.51}$	0.982	0.982
2-Foot Weir.		
$3.048\ lh^{1.45}$	.....	.....
$3.146\ lh^{1.46}$	0.214	0.107
$3.147\ lh^{1.47}$	0.886	0.443
3-Foot Weir.		
$3.242\ lh^{1.46}$	.....	.....

TABLE 17.—Gauge Heights and Discharge for Rectangular Weirs.

Gauge height, in feet.	Discharge of 6-in. weir.	Discharge of 1-ft. weir.	Discharge of 2-ft. weir.	Discharge of 3-ft. weir.
0.00	0.000	0.000	0.00	0.00
0.05	0.018	0.037	0.08	0.13
0.10	0.051	0.103	0.21	0.33
0.15	0.093	0.188	0.38	0.60
0.20	0.142	0.287	0.58	0.91
0.25	0.195	0.398	0.83	1.26
0.30	0.254	0.521	1.08	1.65
0.35	0.318	0.658	1.35	2.08
0.40	0.391	0.801	1.65	2.53
0.45	0.469	0.950	1.95	3.00
0.50	0.551	1.111	2.27	3.50
0.55	0.636	1.282	2.62	4.01
0.60	0.725	1.459	2.98	4.57
0.65	0.818	1.640	3.34	5.13
0.70	0.916	1.830	3.74	5.73
0.75	1.018	2.028	4.14	6.35
0.80	1.125	2.233	4.54	7.00
0.85	1.235	2.448	4.96	7.65
0.90	1.350	2.668	5.41	8.33
0.95	1.465	2.892	5.86	9.03
1.00	1.584	3.125	6.31	9.73
1.05	.....	3.362	6.76	10.45
1.10	.....	3.609	7.25	11.20
1.15	.....	3.860	7.75	11.95
1.20	.....	4.119	8.25	12.70
1.25	.....	4.387	8.75	13.47
1.30	.....	4.664	9.27	14.27
1.35	.....	.....	9.80	15.07
1.40	.....	.....	10.35	15.87
1.45	.....	.....	10.90	16.68
1.50	.....	.....	11.45	17.53
1.55	.....	.....	12.00	18.38
1.60	.....	.....	.....	19.23
1.65	.....	.....	.....	20.11
1.70	.....	.....	.....	21.01
1.75	.....	.....	.....	21.91
1.80	.....	.....	.....	22.86
1.85	.....	.....	.....	23.81
1.90	.....	.....	.....	24.80
1.95	.....	.....	.....	25.80
2.00	.....	.....	.....	26.80
2.05	.....	.....	.....	27.81
2.10	.....	.....	.....	28.86

TABLE 18.—Gauge Heights and Values of C in the Formula, $Q = Clh^{\frac{3}{2}}$, for the Rectangular Weir.

Gauge height.	Length of Weir, in Feet.			
	6-in.	1-ft.	2-ft.	3-ft.
0.05	3.214	3.304	3.571	3.335
0.10	3.838	3.259	3.322	3.481
0.15	3.601	3.236	3.270	3.442
0.20	3.177	3.210	3.163	3.393
0.25	3.120	3.184	3.320	3.360
0.30	3.092	3.171	3.286	3.347
0.40	3.091	3.166	3.261	3.333
0.50	3.116	3.142	3.210	3.299
0.60	3.119	3.139	3.206	3.270
0.70	3.128	3.124	3.192	3.261
0.80	3.145	3.121	3.172	3.261
0.90	3.162	3.125	3.168	3.252
1.0	3.168	3.125	3.155	3.243
1.1	.....	3.128	3.142	3.236
1.2	.....	3.134	3.138	3.220
1.3	.....	3.147	3.127	3.209
1.4	.....	.....	3.124	3.193
1.5	.....	.....	3.115	3.179
1.6	.....	.....	.....	3.167
1.7	.....	.....	.....	3.160
1.8	.....	.....	.....	3.155
1.9	.....	.....	.....	3.156
2.0	.....	.....	.....	3.159
2.1	.....	.....	.....	3.161

TABLE 19.—Coefficients for Weirs with Two Complete End Contractions, for Use in the Formula, $Q = Clh^{\frac{3}{2}}$.

H=Head.	L=Length of Weir, in Feet.									
	6-in.	1-ft.	2-ft.	3-ft.	4-ft.	5-ft.	7-ft.	10-ft.	15-ft.	19-ft.
0.05	3.214	3.304	3.571	3.335	.....	.....	.....	.....	.....	.....
0.10	3.838	3.259	3.322	3.481	3.494	3.494	3.499	3.504	3.504	3.510
0.15	3.601	3.236	3.270	3.442	3.419	3.424	3.424	3.429	3.435	3.435
0.20	3.177	3.210	3.163	3.393	3.376	3.376	3.381	3.386	3.392	3.392
0.25	3.120	3.184	3.320	3.360	3.344	3.349	3.354	3.360	3.360	3.365
0.30	3.092	3.171	3.286	3.347	3.322	3.322	3.333	3.338	3.338	3.344
0.40	3.091	3.166	3.261	3.333	3.285	3.290	3.301	3.306	3.312	3.317
0.50	3.116	3.142	3.210	3.299	3.264	6.269	3.280	3.290	3.295	3.301
0.60	3.119	3.139	3.206	3.270	3.247	3.253	3.269	3.280	3.285	3.290
0.70	3.128	3.124	3.192	3.261	3.231	3.242	3.258	3.274	3.280	3.285
0.80	3.145	3.121	3.172	3.261	3.221	3.231	3.247	3.269	3.274	3.280
0.90	3.162	3.125	3.168	3.252	3.210	3.226	3.242	3.258	3.269	3.274
1.0	3.168	3.125	3.155	3.243	3.199	3.215	3.231	3.253	3.264	3.269
1.1	.....	3.128	3.142	3.236	3.189	3.205	3.226	3.242	3.258	3.264
1.2	.....	3.134	3.138	3.220	3.178	3.194	3.215	3.237	3.253	3.264
1.3	.....	3.147	3.127	3.209	3.167	3.199	3.205	3.231	3.247	3.258
1.4	.....	.....	3.124	3.193	3.156	3.178	3.199	3.221	3.242	3.258
1.5	.....	.....	3.115	3.179	3.151	3.167	3.189	3.215	3.237	3.253
1.6	.....	.....	.....	3.167	3.140	3.162	3.183	3.210	3.231	3.247
1.7	.....	.....	.....	3.160	.....	.....	3.178	3.205	3.226	3.247
1.8	.....	.....	.....	3.155	.....	.....	.....	.....	.....	.....
1.9	.....	.....	.....	3.156	.....	.....	.....	.....	.....	.....
2.0	.....	.....	.....	3.159	.....	.....	.....	.....	.....	.....
2.1	.....	.....	.....	3.161	.....	.....	.....	.....	.....	.....

DISCUSSION

Mr. Pierce.

C. H. Pierce, Assoc. M. Am. Soc. C. E. (by letter).—Certain comparisons of current-meter measurements with weir discharge by standard formulas, made by the writer and other members of the United States Geological Survey in Hawaii during 1911, verify the conclusions reached by the authors in their experiments on weir discharge.

The weirs were on irrigation ditches which offered excellent opportunities for velocity-area measurements, either in flumes or prismatic sections, within short distances of the weirs. The methods of making the velocity-area determinations were those regularly used by the engineers of the Geological Survey, as follows: The total cross-section having been divided into a number of section units, the mean velocity and area of each were measured; the product of the mean velocity and area of each individual section gave the discharge for that section, the total discharge being the sum of those of the individual sections. In most cases the sections were ½ ft. wide, and they always extended vertically from the water surface to the bottom of the channel. The mean velocity of the section was taken as one-half the sum of the means for the verticals on each edge of the section, the mean velocity for the vertical being one-half the sum of the velocities at two-tenths and eight-tenths of the depth. In this way, four determinations enter into the computation of velocity for any individual section.

The velocity measurements were made with small Price electric current meters which had been rated by the United States Bureau of Standards at Chevy Chase Lake, Maryland.

Meter discharge comparisons were made with a large number of weirs, but the measurements in all cases did not cover a sufficient range in stage for a complete rating. The results of comparisons with four typical weirs will be given. These weirs were all much larger than those used by the authors, and may be considered as types of the better class, being well constructed, and designed to meet the requirements of standard weir formulas. Two of the weirs were rectangular; one had a 10-ft. crest and the other nine panels, each 29 in. wide. The other two weirs were of the Cippoletti type, having crest widths of 17 and 25 ft., respectively.

The comparisons covered a range in stage equal to that reached by the weirs under ordinary working conditions.

10-Foot Rectangular Weir.—This was an especially well-constructed weir, with a long approach channel and good end contractions. Its crest and sides were formed by a sharp-edged steel plate set in concrete. The width of the approach channel at the crest level was 23 ft.;

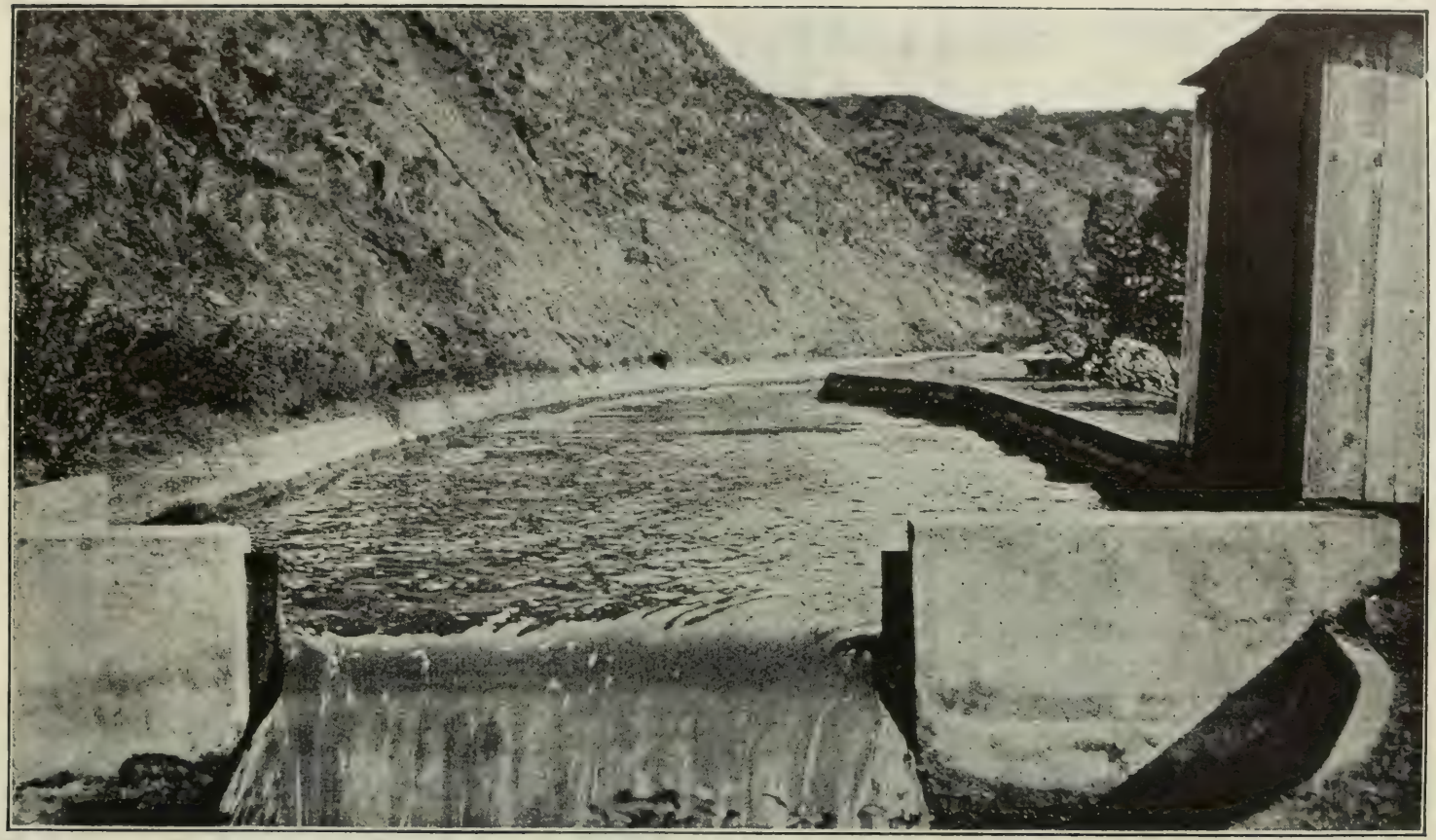


FIG. 4.—TEN-FOOT RECTANGULAR WEIR.

the depth below crest level was 5.3 ft., with side slopes of 1 to 4, giving an approach channel having an area of 115 sq. ft. below crest level. A continuous record of the head on the crest was obtained by an automatic clock register. Mr. Pierce.

Six discharge measurements, taken a short distance below the weir, give a rating curve which is very well defined between heads of 0.4 and 1.6 ft.

A comparison of the discharge curves obtained from the Francis formula,

$$Q = 3.33\ (b - 0.2h)\ h^{\frac{3}{2}}$$

and from current-meter measurements, is shown by Table 20 and by Fig. 5.

Nine-Panel Rectangular Weir.—Each of the nine panels in this weir had a crest width of 29 in. The crests and sides of the panels were formed by sharp-edged steel plates, but it is probable that, at the higher stages, the end contractions were not complete. The pool was approximately circular, and the weir panels were placed on a chord; water was delivered to the pool by a timber flume, 8 ft. wide and 6 ft. deep, the direction of flow of the water on entering the pool from the flume being nearly at right angles to the line of the weir. Here, also, a continuous record of the head on the crest was obtained with an automatic clock register.

Twelve discharge measurements in the flume above the pool give a rating curve which is very well defined between heads of 0.4 and 1.6 ft.

A comparison of the discharge curves obtained from the Francis formula and from current-meter measurements, is shown by Table 20 and by Fig. 5.

25-Foot Cippoletti Weir.—This weir had a sharp-edged crest and sides, formed by a steel plate set in concrete. The crest width was 25 ft. and the end inclinations were 1 to 4. Four current-meter measurements, taken in a lined section of the ditch a short distance above the weir, define a rating curve between heads of 0.4 and 0.9 ft. An automatic clock register gave a continuous record of the head on the weir.

A comparison of the discharge curves obtained from the Francis formula and from current-meter measurements is shown by Table 20 and by Fig. 6. The curves cross at about 0.55 ft. head, showing that, for the lower heads on this weir, the end inclinations should slightly exceed 1 to 4, in order to provide complete compensation for end contractions.* At the higher stages there was some velocity of approach at the center of the weir, diminishing to an inappreciable amount at the sides.

17-Foot Cippoletti Weir.—This was similar in construction to the 25-ft. weir, and likewise had a sharp-edged crest and sides, formed

* Water-Supply Paper No. 200, U. S. Geological Survey, p. 48.

Mr. Pierce.

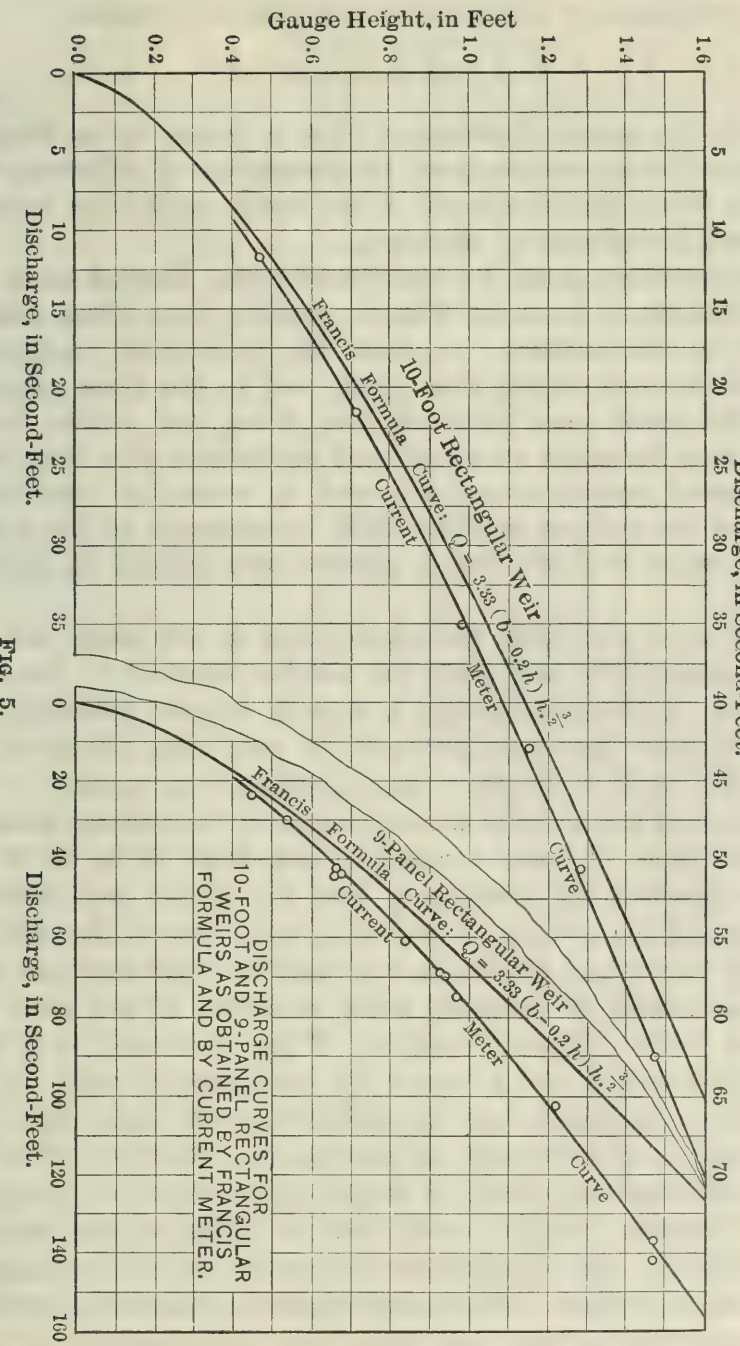
TABLE 20.—COMPARISON OF DISCHARGE BY WEIRS AND BY VELOCITY-AREA MEASUREMENTS.

Average gauge height, in feet.	Variation in gauge height, in feet.	Discharge by Francis formula, in second-feet.	Discharge by current meter, in second-feet.	Difference, in second-feet.	Percentage of difference.
10-FOOT RECTANGULAR WEIR.					
0.471	0.001	10.7	11.7	1.0	9.3
0.716	0.000	19.9	21.5	1.6	8.0
0.984	0.000	31.7	35.0	3.3	10.4
1.156	0.000	40.4	43.0	2.6	6.4
1.282	0.004	47.1	50.6	3.5	7.4
1.474	0.000	57.9	62.7	4.8	8.3
9-PANEL RECTANGULAR WEIR.					
0.45	.....	21.0	23.6	2.6	12.4
0.54	.....	27.4	30.0	2.6	9.5
0.67	.....	37.6	42.9	5.3	14.1
0.67	.....	37.6	43.8	6.2	16.5
0.68	.....	38.4	43.4	5.0	13.0
0.84	.....	51.8	60.7	8.9	17.2
0.93	.....	60.0	69.3	9.3	15.5
0.94	.....	60.8	70.0	9.2	15.3
0.97	.....	63.6	74.9	11.3	17.8
1.22	.....	87.8	103.	15.2	17.3
1.47	.....	113.	138.	25.	22.1
1.47	.....	113.	143.	30.	26.5
25-FOOT CIPPOLETTI WEIR.					
0.405	0.010	21.4	20.9	−0.5	−2.3
0.662	0.007	44.8	46.3	+1.5	+3.3
0.845	0.005	64.7	70.8	+6.1	+8.6
0.852	0.005	65.5	71.8	+6.3	+8.8
17-FOOT CIPPOLETTI WEIR.					
0.37	0.00	12.7	12.3	−0.4	−2.2
0.51	0.00	20.6	19.5	−1.1	−5.3
0.62	0.00	27.6	26.9	−0.7	−2.5
1.12	0.02	67.1	68.5	+1.4	+2.1

by a steel plate set in concrete. The crest width was 17 ft. and the end inclinations were 1 to 4. Four current-meter measurements, taken in an unlined section of the ditch about $\frac{1}{4}$ mile above the weir, define a rating curve between heads of 0.3 and 1.2 ft. An automatic clock register provided a continuous record of the head on the weir.

A comparison of the discharge curves obtained from the Francis formula and from current-meter measurements is shown by Table 20 and by Fig. 6. The curves cross at about 0.8 ft., indicating that, for this weir, the end inclinations should somewhat exceed 1 to 4 for heads less than 0.8 ft., if the effect of end contraction is to be completely

Mr. Pierce.



DISCHARGE CURVES FOR 10-FOOT AND 9-PANEL RECTANGULAR WEIRS AS OBTAINED BY FRANCIS FORMULA AND BY CURRENT METER.

Fig. 5.

Mr. Pierce.

compensated. There was no appreciable velocity of approach at any stage.

General Conclusions.—The results of these comparisons agree with those obtained by the authors as far as showing that, for rectangular weirs, the discharge, as computed by the Francis formula,

$$Q = 3.33\ (b - 0.2h)\ h^{\frac{3}{2}},$$

is less than the actual discharge. This is found to be true for all stages covered by the comparisons, the percentage of difference varying, but usually being greater for the lower heads, except for weirs which were affected by velocity of approach.

The comparisons made by the velocity-area method seem to give a smaller departure from the Francis formula than those made volumetrically by the authors. It must be remembered, however, that the weirs were much larger than those used in the Boise experiments, and that the heads were relatively less, being well within the limits for which weir formulas are considered applicable; also, that the weirs were in actual operation and adjusted to every-day conditions. It appears that the authors extended their experiments on the small 6-in. and 12-in. weirs to heads much greater than should be allowed for those sizes.

For the 17-ft. and 25-ft. Cippoletti weirs at low heads, the Francis formula agrees fairly well with the results obtained by the velocity-area method, the formula giving a slightly greater discharge. For a discharge of more than about 40 sec-ft., in each case, the curves diverge quite rapidly, with the velocity-area discharge the greater.

As a result of comparative measurements by the velocity-area method with several types of weirs varying in size from 12 in. to 30 ft., the writer has reached the conclusion that the design and construction of the weir and the method of delivering the water to the crest greatly modify the theoretical discharge, and that no coefficient can be made to apply accurately to different weirs, although of the same type, if constructed under different designs. What is needed is a standard form of construction and a rating for each type of weir. A certain type should be designed, say, for delivering not more than 1 sec-ft. of water, and a rating should be provided for this weir, with definite limits as to operating head; a second type should be designed for quantities between 1 and 5 sec-ft., with a corresponding rating; and, in like manner, weirs of different capacities should be designed for specified limits of head. The rating should be determined experimentally for each type of weir, and from direct observations on that particular type. A set of standard plans and specifications for every rating would make it possible for different persons to construct weirs,

Mr. Pierce.

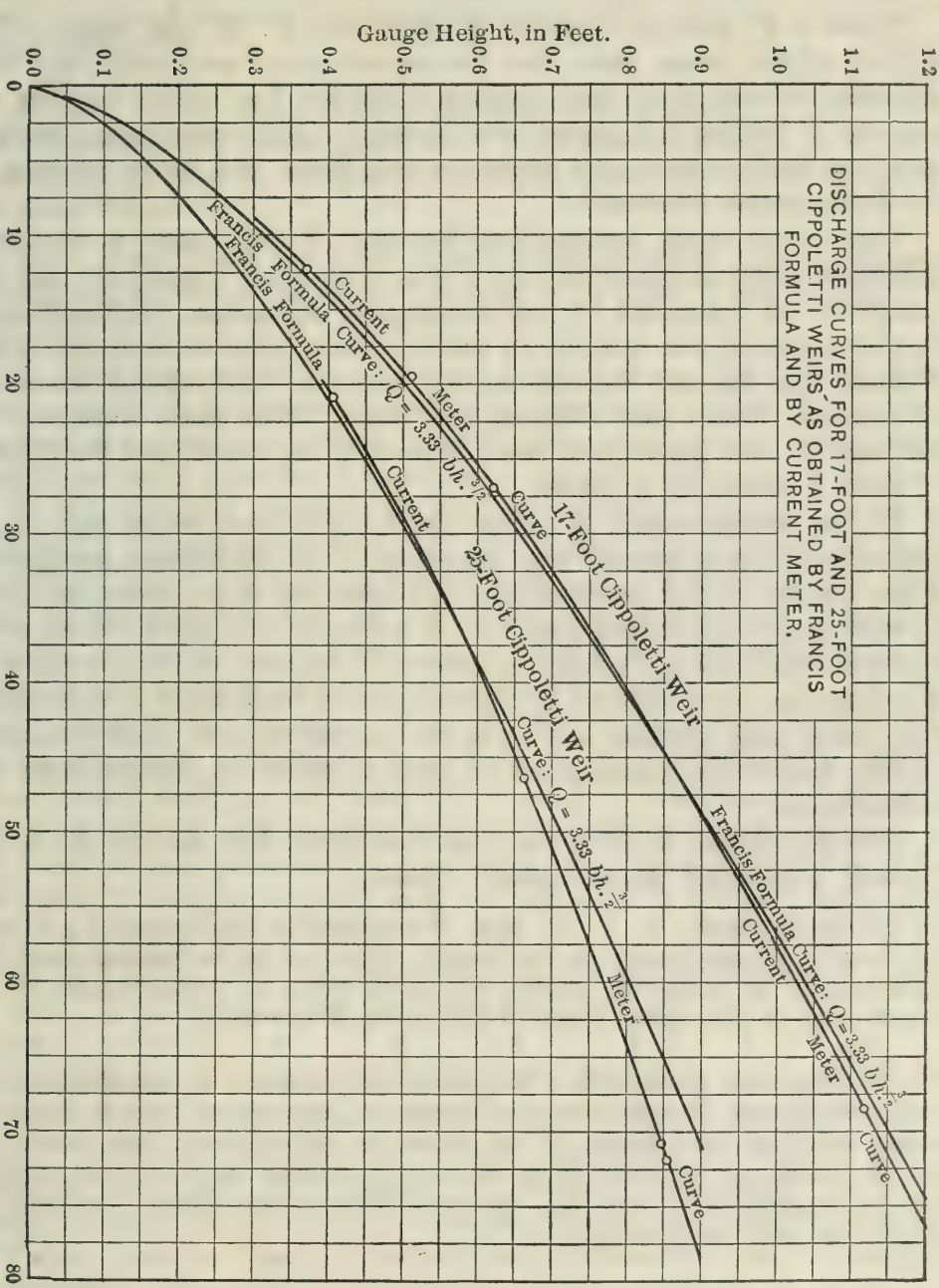


Fig. 6.

Mr. Pierce. and under different conditions, but according to the same design, and giving comparable results. The rating for any weir might be expressed algebraically, but probably not in the simple exponential equation of the Francis formula.

Mr. Longley. FRANCIS F. LONGLEY, ASSOC. M. AM. SOC. C. E. (by letter).—The authors of this paper state that the experiments "were made, not with the idea of obtaining new general formulas for weirs, but for the purpose of testing the general weir formulas and ascertaining whether they are applicable to the weirs on the Boise and other projects of the Reclamation Service."

One of the most conspicuous features of this paper is the great difference that is noted between the measured quantities and the quantities as computed by old established formulas. Without some further analysis, this is likely to leave the false impression in the minds of those who are not well acquainted with the unparalleled researches of Francis, Fteley and Stearns, and Bazin, that their work was all for naught, and therefore a word regarding the recognized limitations of their formulas is in order.

In the measurement of water by the flow over weirs, one of the factors which may become very uncertain if its limitations are ignored is the matter of end contractions. Perhaps the most convenient form of weir is a sharp-edged notch in a vertical thin plate, which gives a stream with the ends fully contracted. Because of its convenience, this form has been studied by several able investigators in connection with other weir studies, and it is interesting to note their comments on the limitations which must be kept in mind in dealing with end contractions.

The late James B. Francis, Past-President, Am. Soc. C. E., in his "Lowell Hydraulic Experiments", states:

"It is assumed * * * that the quantity discharged by a weir is directly proportioned to its length; this, in weirs having complete contraction, is, however, known not to be true, in consequence of the contraction which takes place at the ends of the weir.

* * * * * * * *

"It has been stated that the proposed formula is applicable only to weirs having a considerable length in proportion to the depth of water running over them. It is found by experiment that, when the length equals or exceeds three times the depth, the formula applies; but in lengths less than this in proportion to the depth, the formula cannot be used with safety.

* * * * * * * *

"* * * In weirs of very short length in proportion to the depth, the effect of the end contraction * * * influences the discharge to a certain distance, A, from the end of a weir; if the whole length of the weir is greater than 2 A, the effect of the end contraction is independent of the length; but if the length is less than 2 A, the

whole breadth of the stream is affected in its flow by the end contractions, and, consequently, the proposed formula would not apply. Mr. Longley.

"In practical applications, this will seldom be an inconvenience, as it is nearly always practicable so to proportion the weir that the length may not be less than three times the depth upon it; if, however, there is no end contraction, the proportion of the length to the depth is not material."

Messrs. Fteley and Stearns developed their formula* to represent the discharge over a weir unaffected by end contractions. They conducted some careful experiments on the effect of end contractions, but state that:

"The chief value of these experiments consists in the warning they give that this form of weir should be avoided in cases where the most accurate results are desired."

M. Bazin, in connection with his researches on this subject, mentions, as one of the uncertain elements that must not be disregarded, the lateral contraction, which, though unimportant in weirs of great length, seriously modifies the results in shorter weirs.

In the extensive experiments carried out at the Cornell University Hydraulic Laboratory in 1899, for the United States Board of Engineers on Deep Waterways, weirs without end contractions were used, both for the standard and the experimental measurements, owing presumably to the uncertainty that would probably result with contracted weirs, at the rather high heads that were to be used.

In short, it is generally accepted that uncertainty results from disregarding the recognized limitations of end contractions. Within these limits, data are at hand for making a satisfactory correction for the effect of end contractions. The controlling condition is as set forth in the statement quoted from the "Lowell Hydraulic Experiments," which is in effect that the influence of the end contraction must not extend to the middle of the weir. The law governing the extent of this influence is not known, but Francis considered it unsafe to use a head on the weir greater than one-third the length of the crest.

Disturbance of stream lines sometimes extends to a surprising distance from the source of the disturbance, as note the familiar experiment of the faucet and the knife blade. Open the faucet just enough to get a very thin, but smooth and perfectly continuous stream of water; hold the point of a knife in the stream, well below the faucet, and note how the stream is broken, not only below, but for several inches above the knife.

The formula of Francis, and also that of Cippoletti, which is derived directly from it, are therefore admittedly unsuited for the com-

* *Transactions*, Am. Soc. C. E., Vol. XII, p. 111.

Mr. Longley.

putation of the discharge of weirs with end contractions when the head on the weir is greater than about one-third of the crest.

The authors, however, have recorded a comparison of their results with the results of computations by the Francis and Cippoletti formulas without regard for or comment on this limitation, although the maximum head of each separate series of their observations lies far above this limit.

TABLE 21.

	Length of weir, in feet.	Maximum head on weir, in feet.	Ratio of maximum head to length.
Cippoletti weir	0.5	0.95	1.89
	1.0	1.98	1.98
	2.0	1.99	0.99
	3.0	2.41	0.80
Rectangular weir	0.5	1.00	2.00
	1.0	1.34	1.34
	2.0	1.57	0.79
	3.0	2.06	0.69
Safe limiting ratio, as stated by Francis			0.33

An examination of the data of Francis, Fteley and Stearns, and Flinn and Dyer,* who experimented on the Cippoletti weir, shows that they all keep substantially within this approximate limit.

It is not surprising, therefore, that computations based on the Francis and Cippoletti formulas do not check the observed weir discharges. There are few or no precise data regarding the effect of end contractions on the discharge, when the weir is so short that the influences of the two end contractions overlap. A good idea of this effect may be obtained, however, from an analysis of the authors' data, on the assumption that their short weirs approximated the conditions of rectangular orifices.

Merriman gives the following formula for the rectangular orifice:

$$q = 8.02\, cbd\sqrt{h}$$

in which c is the coefficient of discharge,
b is the width,
d is the depth,
and h is the head on the center of the orifice.

Certain experiments by both Francis and Fteley and Stearns give the ratio of the depth of flow measured directly over the edge of a weir, to the depth measured above the origin of the surface curve, as approximately 0.86, and the depth of the orifice, therefore, may be taken as 0.86 of the observed head.

* *Transactions*, Am. Soc. C. E. Vol. XXXII, p. 9.

From this, for example, compute a few of the quantities for the 12-in. rectangular weir. An examination of Fanning's tables shows that the coefficient of discharge for the conditions assumed does not vary much from 0.61. Mr. Longley.

TABLE 22.

Number of run.	Average gauge height, in feet = h^l.	Assumed depth of orifice, $D = 0.86\,h^l$.	C.	Head on center of orifice, $h = h^l - \frac{d}{2}$.	Quantity, in Cubic Feet per Second.	
					By orifice formula.	By measurement.
1.....	1.338	1.150	0.61	0.763	4.91	4.780
20.....	1.233	1.060	0.61	0.703	4.34	4.292
3.....	1.205	1.037	0.61	0.687	4.20	4.177
4.....	1.130	0.972	0.61	0.644	3.81	3.757
2.....	1.126	0.968	0.61	0.642	3.79	3.743
5.....	1.044	0.898	0.61	0.595	3.39	3.332
6.....	0.982	0.845	0.61	0.559	3.09	3.025
7.....	0.909	0.782	0.61	0.518	2.75	2.695

Obviously, this computation comes a great deal closer to the measured discharge than does the Francis formula so far outside of its recognized limiting conditions.

The formulas the authors have deduced, and the rating curves they have prepared, are of value in computing the discharge of weirs identical with those used in the experiments; but the very diversity of coefficients and exponents which they have set forth in their twenty-one formulas, emphasizes the impossibility of their general application.

The points which are important to keep in mind are that the exhaustive work of Francis on weir measurement was corroborated in a substantial degree by the studies of Fteley and Stearns, and later by the even more extensive researches of Bazin; that the formulas deduced by them have certain recognized limitations, and that, having proper regard for these limitations, the reliability of the formulas cannot be questioned.

W. G. Steward, Esq., and J. S. Longwell, Jun. Am. Soc. C. E. (by letter).—In his discussion Mr. Pierce brings out the point that the results of his experiments agree with those of the writers in that the actual discharge is greater than that given by the Francis formula, but that the velocity-area results show a smaller deviation than the volumetric tests. It should be remembered that Mr. Pierce's experiments were made on very large weirs, and that the departure of the actual discharge from the formula discharge occurs at gauge heights within the limits of accuracy, as generally taken, for the formulas, that is, for depths on the weir of less than one-third of the crest length. The results presented by the writers agree closely with the formulas Messrs. Steward and Longwell.

Messrs. Steward and Longwell.

for gauge heights above the limits of the formula in some cases, and practically to the limits in all cases. The percentages of difference between the writers' results and the formula at gauge heights equal to one-third of the length of the crest are shown in Table 23.

TABLE 23.

Size.	Percentage that the Actual Discharge is in Excess of the Formula at One-third of the Crest Length.	
	Cippoletti weirs.	Rectangular weirs.
6 in.	0	2.75
1 ft.	0	2.03
2 ft.	0.95	2.64
3 ft.	1.98	4.40

It will be noted that these differences are far below those given by Mr. Pierce for points of lower proportional gauge heights.

In the tabulation of the results of the writers' experiments, the discharge curves of the Francis formula and the Cippoletti formula have been used for a comparison of results. No attempt was made to show any discrepancy in these formulas; they were used for making a comparison, up to the limits of the formulas, and to show that, for gauge heights of more than one-third of the length of crest, the formula, as stated by Francis, does not apply. It is quite a common belief that the formulas are correct to almost any depth on the weir, in spite of the statement of Francis to the contrary.

The accuracy of the formulas within the stated limits was brought out quite clearly by the discharge curves, proving that, as long as the depth of water is less than one-third of the crest length, the formulas will give very accurate results.

No attempt was made to develop any formula to apply for all depths, but the results of the actual tests were given and the resulting equations of the discharge curves tabulated. It is to be hoped that some general law of flow for all depths may be worked out by some one.

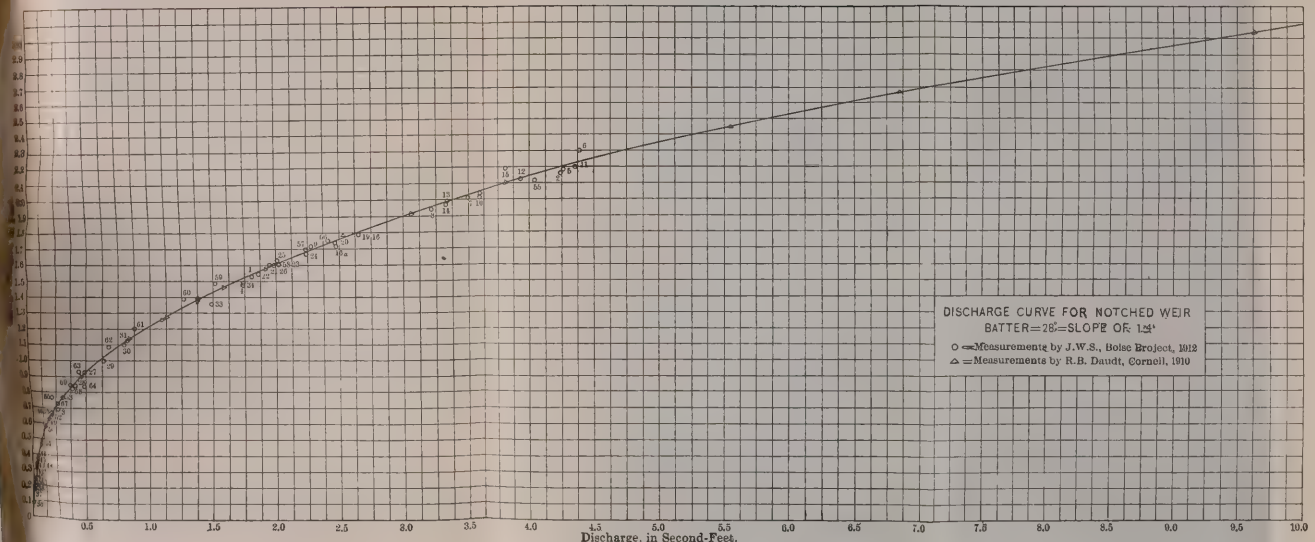
The following conclusions and formulas are presented for consideration. They are based on:

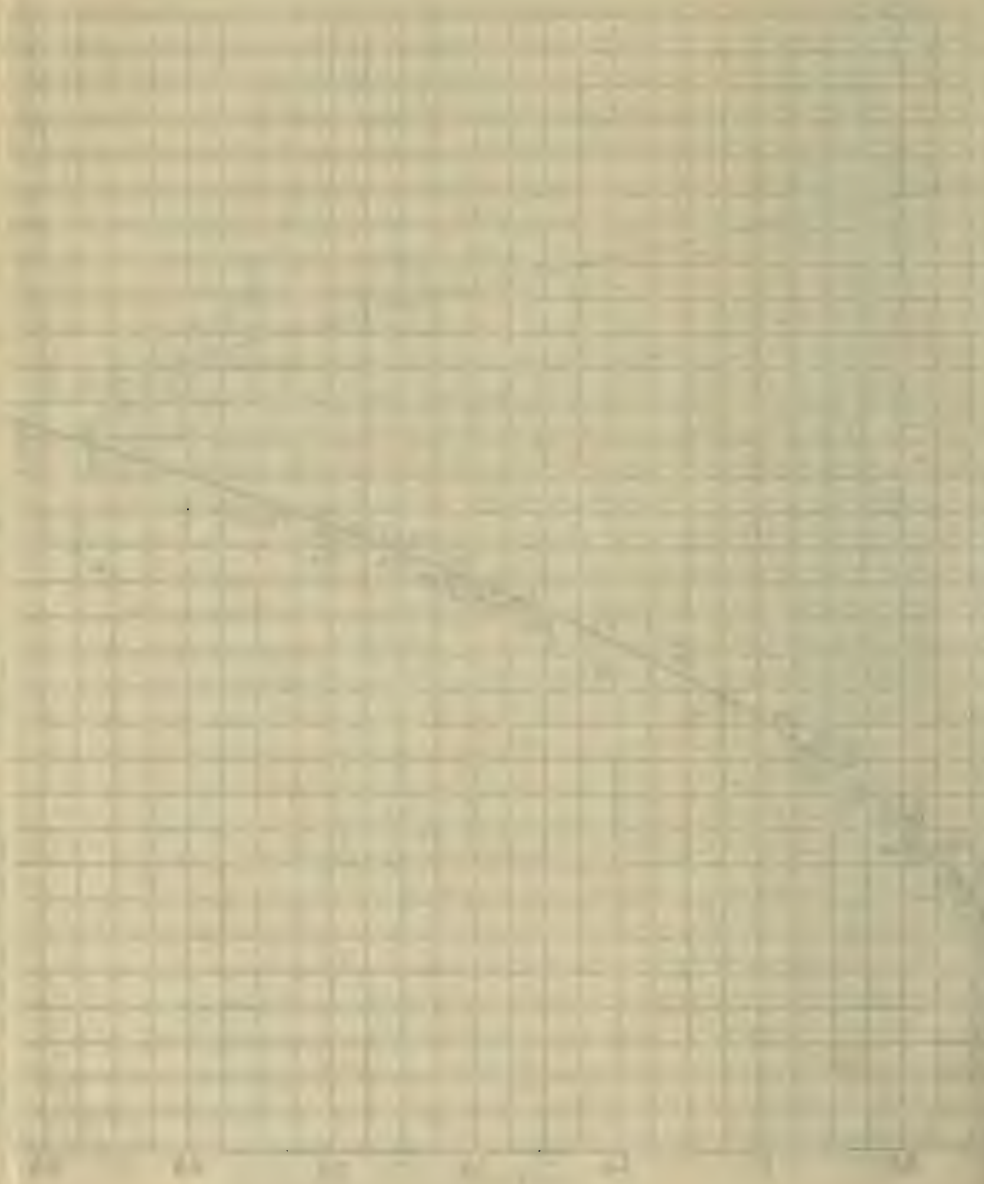
(*a*) The series of weir experiments conducted by the writers on the Boise Project, Idaho, in 1911.

(*b*) The experiments on notched weirs conducted by Mr. J. W. Strohecker, on the Boise Project, in 1912. (See Plate XL and Table 24.)

(*c*) A series of experiments on notched weirs conducted by Ralph B. Daudt, Assoc. M. Am. Soc. C. E. (See *The Cornell Civil Engineer*, March, 1912; see also Plate XL and Table 24.

PLATE XL.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1264.
STEWARD AND LONGWELL ON
WEIR DISCHARGE EXPERIMENTS.





(*d*) A series of experiments on rectangular weirs conducted by Messrs. H. W. Maynard and J. P. Hurley. (See *The Cornell Civil Engineer,* April, 1912.) For a copy of the original notes on this series of experiments the writers are indebted to Mr. Paul Macy, Editor-in-Chief, *The Cornell Civil Engineer.* Messrs. Steward and Longwell.

TABLE 24.—Gauge Heights and Discharges in Experiments Conducted on Boise Project in 1912 by J. W. Strohecker on a Notched Weir; also Gauge Heights and Discharges in Experiments Conducted at Cornell University by R. B. Daudt, Assoc. M. Am. Soc. C. E., in 1910.

Experiments on Boise Project in 1912.												Cornell Experiments.	
No. of run.	Gauge.	Discharge, in second-feet.	No. of run.	Gauge.	Discharge, in second-feet.	No. of run.	Gauge.	Discharge, in second-feet.	No. of run.	Gauge.	Discharge, in second-feet.	Gauge.	Discharge, in second-feet.
36	0.094	0.002	66	0.666	0.224	33	1.36	1.382	19	1.79	2.672	0.4775	0.093
37	0.179	0.005	53	0.677	0.224	60	1.39	1.280	16	1.79	2.672	0.6619	0.214
38	0.195	0.009	3	0.69	0.268	4	1.48	1.734	8	1.945	3.236	0.9025	0.470
35	0.203	0.010	67	0.730	0.265	34	1.485	1.734	14	1.98	3.360	1.142	0.846
40	0.216	0.011	65	0.77	0.208	59	1.49	1.580	13	1.99	3.390	1.259	1.076
39	0.219	0.010	53	0.776	0.309	1	1.535	1.814	7	2.015	3.492	1.272	1.10
47	0.245	0.015	68	0.829	0.372	22	1.55	1.896	10	2.114	3.692	1.377	1.346
42	0.312	0.030	28	0.83	0.392	21	1.592	1.952	55	2.12	4.036	1.3875	1.349
43	0.318	0.034	64	0.84	0.492	26	1.595	1.980	12	2.135	3.936	1.4575	1.549
41	0.333	0.038	69	0.848	0.376	58	1.60	1.896	2	2.160	4.176	1.5855	1.89
48	0.350	0.041	27	0.921	0.492	23	1.608	2.066	18	2.169	4.230	1.7845	2.523
46	0.354	0.045	63	0.93	0.424	25	1.628	2.064	5	2.182	4.248	1.9179	3.087
45	0.38	0.049	29	0.996	0.638	24	1.674	2.236	11	2.185	4.282	2.0357	3.588
44	0.396	0.057	62	1.09	0.676	57	1.70	2.236	15	2.19	3.826	2.1045	3.803
51	0.479	0.094	30	1.094	0.794	9	1.713	2.352	6	2.301	4.392	2.4495	5.56
50	0.578	0.148	31	1.13	0.836	16a	1.72	2.412	....	.....	.....	2.6645	6.85
49	0.63	0.185	61	1.20	0.878	56	1.745	2.392	....	.....	.....	3.0375	9.632
52	0.650	0.204	32	1.22	1.480	20	1.748	2.472	....	.....	.....	.....	.....

Conclusions.

1.—The discharge of a sharp-crested, contracted, rectangular weir, with any given base and gauge height, added to the discharge of a 28° notched weir, with the same gauge height, gives a discharge equal to that of a Cippoletti weir with the same base and gauge height. This is shown on Plate XLI where the combined discharge of a notched weir and rectangular weirs for various lengths of base are plotted, together with the discharges of Cippoletti weirs of equal bases.

2.—The relation of discharges of weirs of similar shapes, within the limits of experiments conducted, is the $\frac{5}{2}$ power of the proportion of bases at the same proportional gauge heights.

Messrs. Steward and Longwell.

FORMULAS.

Let Q_1 = Discharge of a calibrated weir at head, h_1;
Q_2 = Discharge of another weir at head, h_2;
b_1 = Length of crest of calibrated weir;
b_2 = Length of crest of another weir the discharge of which is required;
h_1 = Head on calibrated weir;
h_2 = Head on weir having a base = b_2.

This head is taken so that

$$h_2 = \frac{b_2}{b_1} \times h_1 \text{; that is, } b_1 : h_1 :: b_2 : h_2, \text{ as shown by Fig. 7.}$$

Then Q_1 (at h_1) $\times \left(\frac{b_2}{b_1}\right)^{\frac{5}{2}} = Q_2$ (at h_2) in which $\left(\frac{b_2}{b_1}\right)^{\frac{5}{2}}$ is a constant.

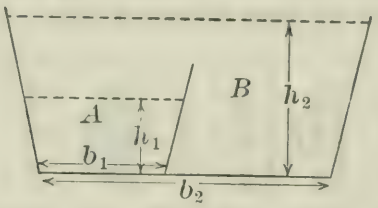


FIG. 7.

Which is derived as follows:

$Q = cbh^{\frac{3}{2}}$, the theoretical formula for discharge of weirs.....(1)

$Q_1 = ch_1 (h_1)^{\frac{3}{2}}$, the discharge of weir A at h_1 (at gauge height).(2)

$Q_2 = ch_2 (h_2)^{\frac{3}{2}}$, the discharge of weir B at h_2 (gauge height as specified above).....(3)

As assumed, $h_2 = \frac{b_2}{b_1} \times h_1$.....(4)

$Q_2 = cb_2 \frac{b_2}{b_1} h_1^{\frac{3}{2}}$. Substituting in Equation 3 the value for h_2 in Equation 4.....(5)

$Q_1 \times cb_2 \left(\frac{b_2}{b_1} h_1\right)^{\frac{3}{2}} = Q_2 \times cb_1 (h_1)^{\frac{3}{2}}$, multiplying Equations 2 and 5.....(6)

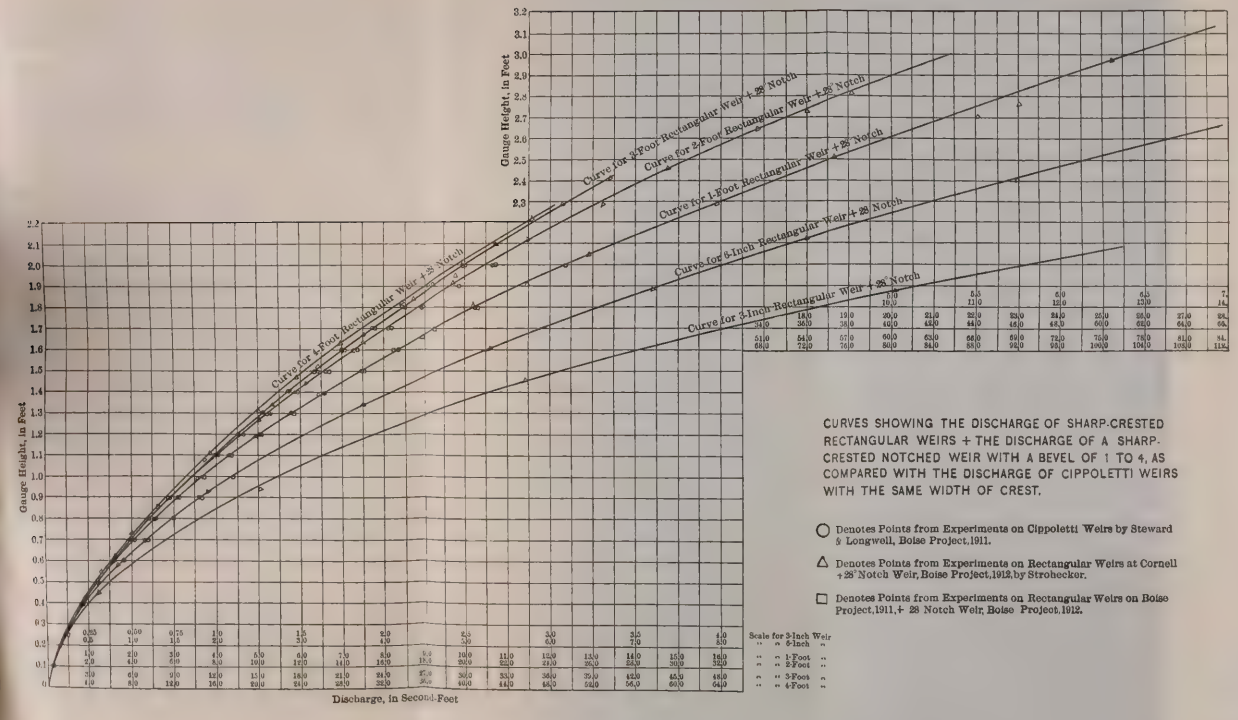
$Q_1 \times \frac{b_2 \times \left(\frac{b_2}{b_1}\right)^{\frac{3}{2}}}{b_1} = Q_2$. Dividing by $cb_1 (h_1)^{\frac{3}{2}}$ and eliminating c and h_1.....(7)

$Q_1 \times \left(\frac{b_2}{b_1}\right)^{\frac{5}{2}} = Q_2$.....(8)

(Conditions being specified above.)

Therefore, having the correct calibration of a weir with a given base and to any given height, the discharge of a weir of similar shape

1092A,

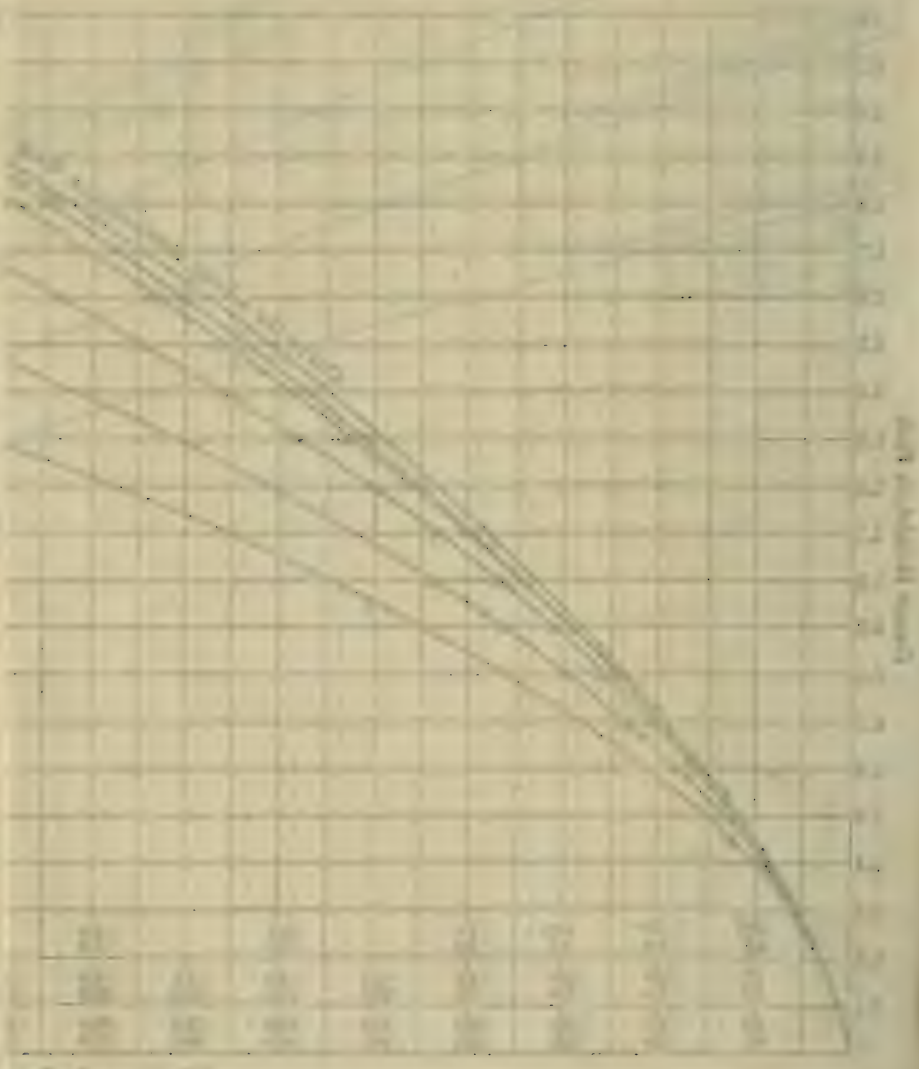

Gauge Height, in Feet
Discharge, in Second-Feet
Curve for 4-Foot Rectangular Weir + 28° Notch
Curve for 3-Foot Rectangular Weir + 28° Notch
Curve for 2-Foot Rectangular Weir + 28° Notch
Curve for 1-Foot Rectangular Weir + 28° Notch
Curve for 6-Inch Rectangular Weir + 28° Notch
Curve for 3-Inch Rectangular Weir + 28° Notch
Scale for 3-Inch Weir
" " 6-Inch "
" " 1-Foot "
" " 2-Foot "
" " 3-Foot "
" " 4-Foot "

CURVES SHOWING THE DISCHARGE OF SHARP-CRESTED RECTANGULAR WEIRS + THE DISCHARGE OF A SHARP-CRESTED NOTCHED WEIR WITH A BEVEL OF 1 TO 4, AS COMPARED WITH THE DISCHARGE OF CIPPOLETTI WEIRS WITH THE SAME WIDTH OF CREST.

○ Denotes Points from Experiments on Cippoletti Weirs by Steward & Longwell, Boise Project, 1911.

△ Denotes Points from Experiments on Rectangular Weirs at Cornell + 28° Notch Weir, Boise Project, 1912, by Strohecker.

□ Denotes Points from Experiments on Rectangular Weirs on Boise Project, 1911, + 28 Notch Weir, Boise Project, 1912.



and with any desired length of base may be computed to a gauge height equal to the proportional lengths of the bases, provided the proportional shapes of forebay, crest, etc., are kept. This formula gives results well within the accuracy of the experiments conducted. Messrs. Steward and Longwell

TABLE 25.—COMPUTED FROM CORNELL EXPERIMENTS ON RECTANGULAR WEIRS, AND U. S. RECLAMATION SERVICE EXPERIMENTS ON NOTCHED WEIRS. THE TABLES GIVE THE DISCHARGE OF RECTANGULAR WEIRS, THE DISCHARGE OF 28° NOTCHED WEIRS, AND THE DISCHARGE OF THE TWO COMBINED, WHICH IS EQUAL TO THE DISCHARGE OF A CIPPOLETTI WEIR, AS SHOWN ON PLATE XLI.

Head.	Rectangular weir.	28° Notch.	Cippoletti weir.	Head.	Rectangular weir.	28° Notch.	Cippoletti weir.
3-INCH CREST.				1-FOOT CREST.			
0.25	0.10	0.02	0.12	0.31	0.48	0.03	0.51
0.45	0.23	0.08	0.31	0.75	1.93	0.29	2.22
0.94	0.74	0.52	1.26	0.985	2.97	0.59	3.56
1.46	1.41	1.44	2.85	1.19	4.01	0.94	4.95
1.88	2.08	2.95	5.03	1.39	5.06	1.38	6.44
2.29	2.88	4.70	7.58	1.66	6.74	2.16	8.90
2.705	3.76	7.73	11.49	1.91	8.36	3.07	11.43
3.08	4.54	9.43	13.97	2.05	9.35	3.55	12.90
.....	....	....	.....	2.29	11.19	4.70	15.89
.....	....	....	.....	2.51	12.70	5.95	18.65
.....	....	....	.....	2.70	14.90	7.20	22.11
.....	....	....	.....	2.76	15.44	7.60	23.04
.....	....	....	.....	2.97	16.18	9.1	25.28
6-INCH CREST.				2-FOOT CREST.			
0.255	0.185	0.02	0.205	0.17	0.045	.....	0.04
0.58	0.66	0.17	0.83	0.55	2.41	0.135	2.54
0.93	1.39	0.51	1.90	0.86	4.80	0.42	5.22
1.34	2.50	1.26	3.76	1.27	8.91	1.09	10.00
1.61	3.30	2.00	5.30	1.63	12.85	2.07	14.92
1.89	4.21	2.99	7.20	2.12	19.11	3.85	22.96
2.13	5.09	3.90	8.99	1.91	16.45	2.97	19.42
2.40	6.18	5.30	11.48	2.29	21.74	4.70	26.44
2.65	7.07	7.20	14.27	2.46	23.90	5.60	29.50
2.87	8.32	8.35	16.67	2.64	26.87	6.80	33.67
3.05	8.73	9.70	18.43	2.73	28.67	7.40	36.07
.....	.....	....	.....	2.81	30.16	8.00	38.16
3-FOOT CREST.				4-FOOT CREST.			
0.145	0.51	....	0.51	0.27	1.84	0.02	0.186
0.402	2.49	0.06	2.55	0.425	3.52	0.07	3.59
0.730	5.72	0.27	5.99	0.625	6.27	0.19	6.46
1.08	10.44	0.75	11.19	0.92	7.17	0.50	7.67
1.34	14.72	1.26	15.98	1.11	14.67	0.80	15.47
1.52	17.45	1.74	19.19	1.31	18.79	1.20	19.99
1.67	20.28	2.20	22.48	1.47	21.79	1.59	23.38
1.84	23.43	2.72	26.15	1.63	25.80	2.06	27.86
1.95	25.98	3.21	29.19	1.82	31.02	2.72	33.74
2.10	28.18	3.88	32.06	1.905	33.02	3.05	36.07
2.29	31.93	4.90	36.83	2.035	36.11	3.58	39.69
2.41	34.63	5.60	40.23	2.22	41.72	4.50	46.22

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1265

SOME EXPERIMENTS WITH MORTARS AND CONCRETES MIXED WITH ASPHALTIC OILS.*

By Messrs. Arthur Taylor and Thomas Sanborn.

With Discussion by Messrs. Logan Waller Page, William J. Boucher, A. T. Goldbeck, and Arthur Taylor.

Object of Tests.

Recently, the attention of engineers has been called to some tests of oil-mixed mortar and concretes,† by Logan Waller Page, M. Am. Soc. C. E., which are held to indicate that oil will mix uniformly with concrete or cement mortar, forming what may be termed an emulsion, and that concretes and mortars, thus mixed with certain percentages of oil, are "absolutely water-tight under pressures as high as 40 lb. per sq. in." and, "under low pressures, both are waterproof."

In order to determine whether these results would hold true for concretes mixed with western (asphaltic) oils, the writers began a series of tests in February, 1912, laying special emphasis on the question of permeability under pressure.

Conclusions.

1.—The following experiments, although incomplete and inconclusive, indicate that, with western (asphaltic) oils, the facts stated by Mr. Page in regard to strength and water-tightness of oil-mixed concretes

* Presented at the meeting of April 16th, 1913.

† "Some of the Properties of Oil-Mixed Portland Cement Mortar and Concrete." *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 255.

and mortars are open to serious question, and should not be accepted in practice until further experiments have demonstrated their truth more fully.

2.—Concrete and cement mortars with 5, 10, and 15% of oil, by weight of cement, were found by these tests to be more permeable than concrete and cement mortars without oil, when subjected to pressures of 20, 40, and 60 lb. per sq. in.

3.—The strength of mortars was found to be affected seriously by the addition of as low as 5% of oil.

4.—The absorption of water by mortars under low heads was found to be but little decreased by the addition of light oils, and increased with heavy oils.

Description of Tests.

Great care was taken to secure uniformity of qualities and quantities in all the experiments.

In the permeability tests, three specimens of each mixture were tested; and in the tensile and compressive tests, either three or six specimens of each kind were used.

The quantity of oil used in each case was determined as a percentage, by weight, of the quantity of cement. It was found, as stated by Mr. Page, that the western (asphaltic) oils will also readily emulsify with wet cement, cement mortar, or concrete, and can become mixed so thoroughly in the mass that on examination with a reading glass no unassimilated globules of oil are visible. The mixture has a rich glossy appearance, the oil, apparently, has spread itself out in a thin film enveloping each particle of sand and rock. Even a very thick oil—one which is so viscous that when cold it cannot be poured—will mix and disappear in concrete or mortar when it is heated and mixed with hot aggregates. The amount of mixing was found to be a principal factor in determining whether or not the oil is incorporated completely and uniformly with the other ingredients of a mortar or concrete.

The consistency of the concrete or mortar also governs the degree of emulsification; the wetter the mixture the better the oil emulsifies.

Materials.

The oils used were commercial products having trade names as indicated. Their mechanical analysis is shown in Table 1.

TABLE 1.—Mechanical Analysis of Oils.

Name.	Baumé gravity.	Flash point.	Burns.	Asphaltum.
Boiler Fuel	17°	174° Fahr.	.....	42%
Richmond Road Oil No. 6	13°	260° "	400° Fahr.	75%
Richmond Fuel	16°	210° "	300° "	52%
Star Fuel	24°	195° "	245° "	21%
Liquid Asphalt	0°.9	.....	.....	92%

Portland cement complying with standard specifications, clean river sand, and two kinds of river gravel were used for the concretes.

One gravel was graded from 1 in. to ¼ in., the other from ¼ in. down. The percentages of voids were: Sand, 33; fine gravel, 37½; coarse gravel, 49. Portland cement and standard sand were used in all mortars. The proportions used for concrete were for an aggregate of maximum density. The mechanical analysis of the aggregate is shown in Table 2.

TABLE 2.—Mechanical Analysis of Aggregates.

Diameter, in inches.	Percentage passing through sieve.
1.00	100
0.50	72
0.25	57
0.19	52
0.06	41
0.03	32
0.022	25
0.012	15

Methods of Mixing.

Concretes.—The proportions of aggregate are by volumes, loose. The quantity of oil is given as a percentage of the weight of the cement. The quantity of water giving a quaky mixture was determined, and used thereafter.

All mixing was done in a box having an approximate volume of 2 cu. ft.

The sand, fine gravel, and cement were mixed in the usual way, and the oil was poured on the wet mass and thoroughly mixed in until it had completely disappeared, as shown by uniform color and absence of oil spots. To this mixture the previously soaked coarse gravel was

then added, and the whole was mixed again. In order to insure uniformity of mixing, the time of each separate operation was fixed and thereafter followed according to schedule.

The most viscous oils, namely, liquid asphalt and road oils, required a larger percentage of water (for the same consistency) than the mixtures in which the lighter oils were used.

Mortars.—The proportioning of mortars was done by weight. Three parts by weight of standard (Ottawa) sand and one part of cement were mixed to normal consistency; the oil was then poured on the mortar, and the whole was mixed until the oil disappeared and the resulting mass was of uniform color. The viscous oils required more mixing than the lighter oils to bring about this condition.

Neat Cements.—Standard methods of mixing the cement paste were used, the oil being poured on the wet paste and mixed as before until the mass was of uniform color. Apparently there is no limit to the quantity of oil that may be used, as far as emulsification is concerned, the cements containing the largest percentages of oil emulsifying as readily as those containing smaller quantities. The oil and wet cement mix together as perfectly as two grades of oil.

Methods of Testing.

For tension and compression tests, the usual standard methods were used. The permeability tests were made with apparatus similar to that used for such tests at the University of Wisconsin. The apparatus consists of four lengths of $\frac{3}{4}$-in. pipe tapped into a horizontal main. Each of these pipes connects with an 18-in., glass water-gauge fitted with a calibrated scale for measuring the quantity of water between certain heights shown on the gauge. On the end of each vertical pipe there is a union for connecting the specimen holder. These holders are of 6-in., threaded, wrought-iron pipe, 11 in. long, fitted with cast-iron caps which screw down on a rubber gasket, thus making a water-tight joint. A $\frac{3}{4}$-in. pipe from this cap connects with the union previously mentioned.

After the specimens are connected up, water is let in from the top, through a valve on the horizontal pipe, until it stands at a certain height on the gauge glass. The pressure is applied by compressed air, which is kept constant by two supply tanks and is gauged at its entrance to the apparatus.

A valve is placed between each specimen and the horizontal main, so that a leaky or broken specimen can be shut off without disturbing the test of the other specimens.

The permeability specimens were made in the holders previously mentioned, Fig. 1. The concrete specimens were 6 in., and the mortar specimens 3 in., thick. Before the concrete or mortars were put in place, a ½-in. lining of neat cement was plastered on the inside of the pipe, which insured a perfectly tight bond.

All specimens stood dry for 24 hours after mixing; water was then put on top of the specimen until the test was made.

The top and bottom surfaces of each specimen, before being attached to the testing machine, were chipped uniformly by hand tools, in order to provide for each specimen a surface as nearly uniform as possible.

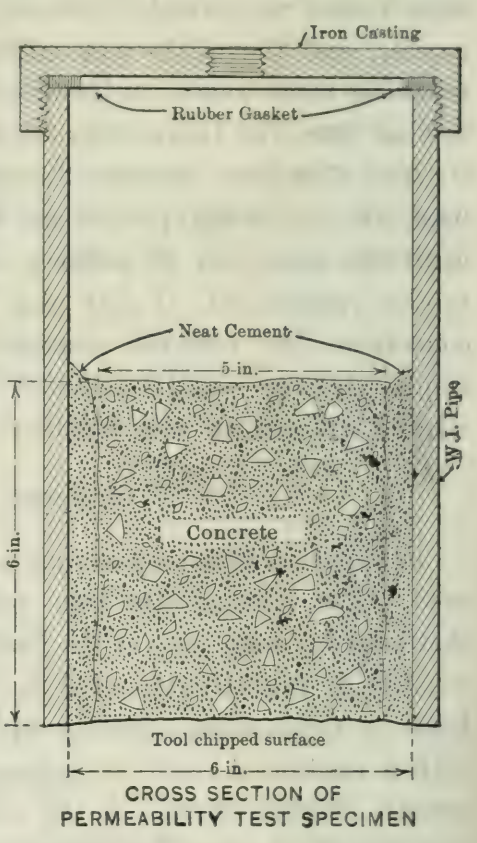


FIG. 1.

RESULTS OF TESTS.

Time of Set.—It was found that, with oil-mixed specimens, the time of setting was retarded, in both the initial and final set. These tests were made with the Vicat needle according to the Standard Specifications. The oil used was an average middle-grade oil. Table 3 gives the time of initial and final set for various percentages of oil.

TABLE 3.—TIME OF INITIAL AND FINAL SET.

Percentages of oil.	Initial set, in hours.	Final set, in hours.
0	4.	5.20
5	5.20	6.
10	5.80	7.30
15	6.80	9.50
25	12.20	36.

These results confirm the observations of Mr. Page.* A theory may be advanced that the time of setting is retarded and the strength is decreased, due to the fact that the oil film separates particles of cement from each other, so that there is not the complete intimate association of every particle of cement, as there is in a mixture without oil.

Tensile Strength.—Either three or six specimens of each kind were broken. All tests were made on specimens set 24 hours in moist air, the remainder of the time under water. Table 4 gives the strength and relative values of specimens having various percentages of oil.

TABLE 4.—Tensile Strength of 1:3 Mortar.

Averages of three or more briquettes.

Percentage of oil.	7 Days.		28 Days.	
	Pounds per square inch.	Relative strength.	Pounds per square inch.	Relative strength.
		Percentage.		Percentage.
0	324	100	430	100
5 Boiler Fuel	260	80.3	335	78
10 "	192	59.3	303	70.5
15 "	170	52.5	225	52.4
25 "	137	42.3	213	49.6
Richmond Fuel, 10	170	52.5	273	63.5
Road No. 6, 10	146	45.0	184	42.8
Liquid Asphalt, 10	141	43.5	220	51.2

Crushing Strength.—All compressive specimens were made of 1:3 mortar in 3-in. cubes. Specimens stood 24 hours in moist air; the remainder of the time before testing they were under water. All tests were made on an electrically-driven machine, the load being applied slowly. In every test the specimens failed regularly, the sides sloughing off uniformly. In Table 5, are given the compressive strengths and relative values of oil-mixed specimens compared with plain or no-oil mortar.

Absorption.—Mortars containing percentages of middle-grade oil up to 25 were slightly less absorptive than plain mortars, the absorption decreasing with the increase in the percentage of oil. The specimens containing heavier oils showed an increase in absorption.

* *Loc. cit.*, p. 258.

It was found that at the end of 24 hours the mortars had reached just about complete absorption. At the end of 72 hours the weight had only increased a negligible fraction over that at the end of 24 hours.

TABLE 5.—28 AND 50-DAY COMPRESSION TESTS: 3-INCH CUBES OF 1:3 MORTAR.

Percentage of oil.	28 Days.		50 Days.	
	Pounds per square inch.	Relative value.	Pounds per square inch.	Relative value.
		Percentage.		Percentage.
0.....	3 950	100	4 400	100
5 Boiler Fuel.....	2 435	61.6	3 620	82.4
10 "	1 780	45.1	2 460	56
15 "	1 460	37.0	2 000	45.5
25 "	712	18.2	1 000	22.8
Richmond Fuel, 10..	1 640	41.5	.....	.....
Road Oil No. 6, 10...	1 235	31.2	.....	.....
Liquid Asphalt, 10..	1 080	27.4	.....	.....

Table 6 gives the results of the absorption tests.

TABLE 6.—RESULTS OF ABSORPTION TESTS, WITH 1:3 MORTAR, 35 DAYS OLD.

	Boiler fuel oil.					Richmond Fuel.	Road Oil No. 6.	Liquid Asphalt.
Percentage of oil, weight in grammes....	0	5	10	15	25	10	10	10
Weight after heating.....	140	131.7	142	135	134	199	204	139
Weight after 24 hours in water.....	143.5	134.3	145.2	137.5	136	205.8	110.5	144.5
Weight after 72 hours in water.....	negligible gain over 24 hours immersion.							
Gain in weight in 24 hours.....	3.5	2.5	3.2	2.5	2	6.8	6.5	5.5
Percentage of gain in water in 24 hours..	2.5	1.97	2.26	1.85	1.49	3.42	3.19	3.96

Permeability Tests.—The permeability tests were made with the specimens heretofore described; at the end of 20 days with concrete, and 10 days with mortar. Three of each were tested, and the average value was taken, except in some cases when the specimens proved to be faulty. Water pressure was applied on concrete for periods of 4 hours each under pressures of 20, 40, and 60 lb. per sq. in., and finally a 4-day test under 15 lb. per sq. in. was run. With mortar specimens, 10- and 20-lb. pressures were applied for 4 hours under each pressure. Tables 7, 8, and 9 give the number of cubic centimeters of water passed through the specimens under the various pressures.

TABLE 7.—Permeability Tests of 1:3:4 Concrete.
All specimens under pressure for 4 hours.

Set.	Kind of oil.	Percentage of oil.	Cubic Centimeters of Water Passed Through.		
			20 lb. per sq. in.	40 lb. per sq. in.	60 lb. per sq. in.
*A**	None	0	64	80	83
A	Boiler Fuel	5	137	114	108
A	" "	10	119	72	53
A	" "	15	66	62	38
B	None	0	32	41	60
B	Boiler Fuel	5	143	128	160
B	" "	10	68	65	....
B	" "	15	87	119	150
C	None	0	40	41	105
C	Boiler Fuel	5	42	53	150
C	" "	10	47	43	110
C	" "	15	38	40	108
	Average of *B* and *C*.†				
		0	36	41	82
	Boiler Fuel	5	42	53	155
	" "	10	57	54	95
	" "	15	62	79	129
D	None	0	10	12	12
D	"	0	9	10	10
D	"	0	11	10	9
E	Richmond Fuel	10	24	31	41
E	" "	10	20	25	28
E	" "	10	29	37	44
F	Road Oil No. 6	10	68	85	75
F	" "	10	56	70	53
G	Liquid Asphalt	10	22	23	33
G	" "	10	21	23	32
G	" "	10	23	25	38

* Set *A* stood dry 24 hours before testing. As the results for increasing pressure do not agree with *B* and *C*, they have been omitted in taking the average.

† *B* for 5% was thrown out as defective.

TABLE 8.—Four-Day Permeability Tests, 1:3:4 Concrete Under Pressure of 15 lb. per sq. in. on Best Average Specimen of Each Set.

Kind of oil.	Percentage of oil.	Cubic centimeters of water passed through.
No oil	0	38
Richmond Fuel	10	62
Road oil No. 6	10	87
Liquid Asphalt	10	185

TABLE 9.—PERMEABILITY TESTS, 1:3 MORTAR.
Average of three specimens. Each pressure on 4 hours.

Kind of oil.	CUBIC CENTIMETERS OF WATER PASSED THROUGH.	
	10 lb. per sq. in.	20 lb. per sq. in.
No oil	0	1
Star Fuel	4.0	4.6
Richmond Fuel	19	11.5

SUMMARY OF CONCLUSIONS.

The following conclusions are drawn from the results of these investigations with asphaltic oils:

1.—Oil-mixed concrete containing 5, 10, or 15% of oil, by weight of cement, is more permeable under pressures from 20 to 60 lb. per sq. in. than concrete without the incorporated oil, and oil-mixed mortar containing 10% of oil is more permeable than plain mortar under pressures of 10 and 20 lb. per sq. in.

2.—Oil-mixed mortars containing percentages of oil up to 25 show slightly less absorption of water than plain mortar, except that with the most viscous oils the mortar becomes more absorptive, and absorption is not an index of the permeability. The absorption decreases with the increase in the quantity of oil.

3.—The tensile strength of oil-mixed mortar is decreased considerably below that of plain mortar. The strength decreases with the increase in the quantity of oil.

4.—The compressive strength of oil-mixed mortars follows lines similar to those of the tensile strength, but with a greater decrease in strength with the quantity of oil.

5.—The strength decreases with the viscosity of the oils, and with the most viscous oils it requires considerably more water to keep a mixture at normal consistency than is required in the fluid oil mixtures.

6.—The relative decrease in strength with the increase in the quantity of oil in mortars is less in 50 than in 28 days.

The writers wish to acknowledge their indebtedness to Professor C. B. Wing, M. Am. Soc. C. E., for his aid and advice in the work, and to the Standard Oil Company of San Francisco for its courtesy in furnishing the oil samples.

DISCUSSION

Mr. Page.

LOGAN WALLER PAGE, M. AM. SOC. C. E. (by letter).—Since reporting the results of his oil-concrete investigations* in September, 1911, additional data have been obtained by the writer which will be of interest at the present time in showing the effect of different types of oils on the strength of oil-mixed concrete and mortar at the age of 2 years. These results, some of which have already been reported at an early age, are given in Tables 10, 11, and 12.

Referring first to Table 10, Tensile Strengths, it will be noted that Oils Nos. 4923, 4981, and 5053 have no deteriorating effect on the tensile strength, at the end of 1 year in the case of No. 5053, or at the end of 2 years in the cases of Nos. 4923 and 4981. Oil No. 5856 more nearly resembles those used by Messrs. Taylor and Sanborn than any of the others, and the effect on the strength is not greatly different from that reported by them. The few tests made with Oil No. 6536, at the end of 28 days, show an increase in the tensile strength with the addition of this oil. It would seem from these results, combined with those of Messrs. Taylor and Sanborn, that some types of oils might be seriously detrimental to the strength of mortars, and, therefore, that their selection should be made from the results of tests of mortar mixed with the oil under consideration.

TABLE 10.—TENSILE STRENGTH OF OIL-MIXED BRIQUETTES STORED IN WATER.

1 : 3 Ottawa Sand.

Mark.	Percentage of oil.	Character of oil.	7 days.	28 days.	6 months.	1 year.	2 years.
16	0	Fluid residual oil No. 4 923	256	296	326	326	293
17	2½	"	299	400	449	320	257
18	5	"	287	316	372	353	...
19	10	"	252	331	360	343	315
20	15	"	268	356	308	327	320
21	20	"	250	335	320	340	333
152	0	Fluid residual oil No. 4 981	283	367	312	...	250
158	2½	"	259	327	323	...	296
163	5	"	268	341	321	...	328
160	10	"	264	329	319	...	265
213	0	Fluid residual oil No. 5 053	244	331	302	288	...
216	2½	"	297	353	298	286	...
217	5	"	313	334	325	309	...
222	10	"	304	371	330	320	...
579	0	Asphaltic petroleum No. 5 856	226	343	348	340	...
580	2½	"	238	297	278	333	...
581	5	"	204	295	323	295	...
582	10	"	161	225	276	285	...
733	0	Asphaltic petroleum No. 6 536	...	347	...	...	...
734	5	"	...	385	...	...	...
735	10	"	...	370	...	...	...

* *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 255.

Mr. Page. The results on crushing strength given in Table 11, and extending up to a period of 2 years, show, in general, that oil-mixed concrete gains strength with age, a pretty strong indication that, with the small quantity of oil incorporated, the concrete is not likely to suffer future deterioration. The ratio of the strengths, however, is not the same with different kinds of oils. When 10% of Oil No. 4981 is mixed with concrete, the strength is cut down to about one-half of that of the plain mixture at all periods up to 2 years. Oil No. 4923, however, does not have this severe effect in decreasing the strength; and, in general, with a very dusty crushed gneiss aggregate, the crushing strength with 10% of oil is about 75% of that of the plain concrete, and with 5% of oil is about 85% of the plain concrete strength. With a well-graded, clean, gravel aggregate, excellent results were obtained with 2½, 5, and 10% of Oil No. 4923, the 5% mixture having 93% and the 10% mixture 90% of the strength of plain concrete. The results obtained at the end of 1 year, on the compressive strength of mortar mixed with five different kinds of oils, and reported in 1911, do not show the serious decrease in strength obtained by Messrs. Taylor and Sanborn with the California asphaltic oils.

Regarding the question of absorption, it is to be regretted that the authors have not explained thoroughly their absorption tests, particularly with regard to the preliminary treatment of their specimens in securing complete dryness. It has been the writer's experience, and likewise that of other investigators, that a 1:3 mortar which will absorb only 2½% of its weight during an immersion of 24 hours, is a rare curiosity. With all the oils that the writer has tried for absorption, he has obtained results the opposite of those reported by Messrs. Taylor and Sanborn. It has been his experience that the heavy oils are more effective as damp-proofers than the lighter oils, when properly mixed. The difficulty of mixing an oil with concrete increases with its specific viscosity, and it is unwise to use an oil with a higher specific viscosity than 45.

The authors' permeability results are not a surprise to the writer, considering the early age at which the tests were made. As time goes on, the oil-mixed specimens, under high pressures, as well as the plain specimens, become impermeable, but, at early periods, oil-mixed concrete, under high pressure, is no more impermeable than plain concrete, although the writer has succeeded, even at early periods, in making the oil mixture of concrete more impermeable than the plain mixture, by using a very carefully graded gravel aggregate. Oil-mixed mortars seem to be, in general, impermeable. The writer would like to make it clear that he considers the addition of oil to the concrete mixture of benefit as a damp-proofer rather than as a means for preventing the percolation of water under great pressure; and not only his laboratory tests, but the results from numerous con-

Mr. Page.

TABLE 11.—Crushing Strength of Oil-Mixed Concrete.

Specimens 6 *c* to 12 *c* are of 1:3:6 concrete.— Crushed gneiss aggregate.

" 13 *c* " 18 *c* " " 1:3:5 " — " " "

" 164 " 224 " " 1:1.5:4.5 " — " " "

" 479 " 482 " " 1:2:4 " — Gravel aggregate.

	Mark.	Percentage of oil.	Character of oil.	Stored in Air.				Stored in Water.			
				28 days.	6 months.	1 year.	2 years.	28 days.	6 months.	1 year.	2 years.
	6 *c*	0	.....	1 020 (3)	980 (2)	1 090 (3)	1 225 (2)	1 370 (2)	1 955 (2)	2 170 (3)	2 205 (2)
	7 *c*	2½	Fluid residual oil No. 4 923.....	750 (2)	915 (2)	1 165 (2)	.....	1 253 (3)	1 715 (2)	1 580 (3)	1 900 (2)
	8 *c*	5	"	711 (2)	975 (2)	1 015 (2)	1 560 (2)	1 142 (2)	1 690 (2)	1 835 (2)	1 865 (2)
	9 *c*	10	"	656 (2)	755 (2)	850 (1)	1 230 (1)	961 (3)	1 400 (2)	1 440 (3)	1 585 (2)
	10 *c*	15	"	542 (2)	670 (2)	675 (2)	.....	750 (2)	995 (2)	1 145 (3)	.....
	12 *c*	25	"	605 (3)	678 (2)	.....	1 005 (2)	620 (3)	1 030 (2)	1 120 (2)	.....
	13 *c*	0	.....	920 (2)	998 (3)	1 010 (1)	1 100 (2)	1 342 (3)	1 705 (3)	1 890 (3)	2 300 (2)
	14 *c*	2½	Fluid residual oil No. 4 923.....	787 (2)	945 (2)	995 (3)	1 010 (2)	1 250 (2)	1 830 (3)	2 410 (3)	2 250 (2)
	15 *c*	5	"	720 (2)	822 (2)	912 (3)	940 (2)	1 082 (2)	1 626 (2)	1 715 (3)	1 805 (2)
	16 *c*	10	"	698 (2)	965 (2)	975 (2)	1 010 (3)	1 040 (2)	1 406 (2)	1 490 (3)	1 480 (3)
	17 *c*	15	"	693 (2)	828 (2)	888 (3)	888 (2)	1 040 (2)	1 380 (2)	1 382 (3)	1 680 (3)
	18 *c*	20	"	572 (2)	639 (2)	772 (2)	672 (2)	904 (2)	1 116 (3)	1 230 (3)	1 130 (2)
	164	0	.....	1 880 (2)	2 050 (2)	2 365 (2)	.....	2 185 (2)	2 670 (2)	2 950 (2)	2 870 (2)
	168	2½	Fluid residual oil No. 4 981.....	1 345 (2)	1 490 (2)	1 560 (1)	1 855 (2)	1 795 (2)	2 330 (2)	2 250 (2)	2 700 (2)
	169	5	"	1 250 (2)	1 145 (2)	1 768 (2)	1 585 (2)	1 925 (2)	2 355 (2)	2 750 (2)	2 800 (2)
	177	10	"	980 (2)	845 (2)	1 170 (2)	.....	1 065 (2)	1 485 (2)	1 625 (3)	1 568 (3)
	215	5	Fluid residual oil No. 5 053.....	622 (2)	846 (2)	830 (3)	847 (2)	1 250 (2)	1 570 (2)	1 627 (3)	1 820 (3)
	221	10	"	841 (2)	798 (2)	1 015 (2)	.....	1 164 (2)	1 590 (2)	1 768 (2)	1 710 (2)
	224	15	"	643 (2)	752 (2)	800 (2)	.....	1 320 (2)	1 710 (2)	1 730 (2)	1 535 (2)
Gravel	479	0	.....	.....	.....	.....	.....	1 920 (3)	3 280 (3)	3 350 (3)	.....
Gravel	480	2½	Fluid residual oil No. 4 923.....	.....	.....	.....	.....	2 420 (2)	3 430 (2)	3 390 (2)	.....
Gravel	481	5	"	.....	.....	.....	.....	2 750 (3)	3 270 (3)	3 110 (2)	.....
Gravel	482	10	"	.....	.....	.....	.....	2 160 (3)	2 820 (2)	3 020 (3)	.....

Mr. Page.

structions, point to the efficiency of the oil as a damp-proofing agency. Oil-mixed mortar, however, might be used with benefit to prevent the percolation of water under pressure, as well as to damp-proof.

TABLE 12.—Analysis of Oils.

Sample number	5 856	6 536
Type	Asphaltic petroleum	Asphaltic petroleum.
Character	Sticky, viscous fluid, with gasoline odor.	Slightly sticky fluid, with naphtha odor.
Specific gravity at 25°/25° cent.	0.945	0.945
Loss at 163° cent., 5 hours, 20 grammes.	30.84	21.76
Character of residue	Soft, sticky, ductile, with slight flow	Soft, sticky, semi-solid, crystalline appearance on surface.
Soluble in CS_2, air temperature	99.93	99.97
Organic matter insoluble	0.07	0.03
Percentage of total bitumen insoluble in 86° paraffin naphtha	7.71	14.42
Fixed carbon	4.22	7.57
Specific viscosity, Engler at 25° cent.	50.0	15.0

The writer has had very successful results with oil-mortar parging applied to the exterior, and in some cases to the interior, of basement walls, with oil-concrete roofs and walls, in the United States Treasury vaults, and in basement floors. He has also had reports regarding oil-mixed concrete structures that were perfectly waterproofed when subjected to heads of 10 ft. Oil-mixed concrete has likewise been used for the construction of pumping pits in very wet soil. Similar constructions of plain concrete in the vicinity were always wet, whereas the oil-concrete constructions remained perfectly dry. The ease with which the oil can be incorporated with the concrete, and the comparative cheapness of the process, make it a desirable one for use in structures subjected to low heads. The following specification for oil is given as one which, it is believed, will give effective results as a damp-proofing agent when used in a concrete or mortar mixture:

Specification for Oil to be Used in Oil-Cement Concrete.

(1) The oil shall be a fluid residual from the distillation of petroleum, and shall contain no admixture of fatty or vegetable oils.

(2) It shall have a specific gravity of not less than 0.920 at a temperature of 25° cent.

(3) It shall show a flash point of not less than 150° cent. by the closed-cup method.

(4) When 240 cc. of the oil are heated in an Engler viscosimeter to 50° cent., and maintained at that temperature for at least 3 min., the first 100 cc. which flows out shall show a specific viscosity of not less than 15 nor more than 30.

In conclusion, it would seem to the writer that, should a duplication of the tests by Messrs. Taylor and Sanborn give the same results

with the California oils as those already obtained, they would be dangerous and valueless oils to use. The writer cannot help being a little doubtful of the reported absorption tests, as they are at variance with those of a number of investigators. He regrets that his own experiments have not included the California oils, but rather the semi-asphaltic product. He hopes, however, to be able to duplicate the authors' tests with the asphaltic oils, in order to throw additional light on their application to oil-concrete mixtures. Mr. Page.

Regarding the effect of drippings of oil on concrete, it must be acknowledged that in a great many instances the concrete mixtures have suffered great deterioration. This, however, is an entirely different proposition from that of mixing oil in very small percentages with wet concrete. A great many lubricating oils have vegetable and animal oil admixtures, and such oils, it is known, cause the destruction of concrete, the lime in the cement causing a saponification of the oil. Moreover, in such cases, as the oil is concentrated on a small surface of concrete, its effect is very serious. In the case of the oil admixture, only a very small percentage of oil is used, and this is distributed throughout the mass.

William J. Boucher, Assoc. M. Am. Soc. C. E.—The following brief remarks do not relate to the asphaltic or other heavy oils, but to the lighter varieties, such as kerosene. Mr. Boucher.

In the period just preceding the opening of the present Rapid Transit Railroad, in New York City, barrels of kerosene were kept at convenient locations, from which the lanterns of the watchmen were filled. One night a barrel leaked, and in the morning an oily spot on the concrete roadbed showed where the kerosene had disappeared. Considerable discussion followed as to whether damage would result, and on account of the layers of asphalt and felt water-proofing about 10 in. below the surface of the concrete, which it was believed would surely be damaged by the kerosene, the concrete was dug up, the water-proofing relaid, and new concrete placed. It has always been a matter of interest to the speaker as to what would have happened to the kerosene-soaked concrete if it had been left in place. Probably on a smooth troweled surface, such as a sidewalk or station platform, it would run off and evaporate, but would it cause deterioration if allowed to remain in ordinary concrete?

On looking up some references on oil mixed with concrete, the speaker found a very interesting article,* giving the experiments and part of the conclusions of James C. Hain, Assoc. M. Am. Soc. C. E., who conducted quite an elaborate series of tests on cement briquettes, treated with or immersed in lard oil, whale oil, castor oil, boiled lin-

* *Engineering News*, March 16th, 1905.

Mr. Boucher.

seed oil, crude petroleum, and signal oil (the latter a mixture of animal fat and mineral oil). Mr. Hain's account of the tests covers more than three pages, and he summarizes as follows:

"At the outset we failed to find [in railroad or industrial works] a specimen of concrete that we felt was positively disintegrated by oil, though we not infrequently heard of it. * * * On the other hand, plenty of concrete was found that oil had penetrated but not disintegrated. Further, our tests on comparatively new briquettes, which were weakened by exposure to air, showed that, with one exception, they were disintegrated by all classes of oils and fats. These oils and fats when arranged according to their effect (as far as our experiments indicate), are as follows: 1, animal fat; 2, animal oil; 3, vegetable oil; and 4, mineral oil. * * * On the following points we are reasonably certain: (1) Most oils penetrate concrete mortar, which makes them dangerous. (2) Concrete is more liable to be disintegrated when saturated with oils and fats if not thoroughly set. (3) A good quality of concrete is less susceptible to the effect of oil than a poor quality, such as a porous, frosted, lean, poorly mixed, or improperly seasoned concrete."

In the issue of April 20th, 1905, of the same periodical, Maximilian Toch, Assoc. Am. Soc. C. E., comments on Mr. Hain's experiments, and says:

"We found that the disintegration [of the briquette immersed in signal oil] was due to the formation of oleate and stearate of calcium. To reduce this to its simplest expression, the animal oils contain acids which combine with the lime, and crystals of stearate and oleate of lime are formed. It is very likely that these crystals in the process of formation have increased the bulk in the briquette and the bond which has been formed by the lime in the set cement has been totally disintegrated and ruptured.

* * * * * *

"Machinery oil which contains no animal fat is evidently harmless to cement concrete for the reason that no combination can occur. Machinery oils are almost exclusively paraffine oils."

Mr. Goldbeck.

A. T. GOLDBECK,* ESQ.—It is a peculiar thing that, with all the oils used in Mr. Page's experiments, the tensile strength of 1:3 mortar has been but very slightly affected by the oil admixture. In a great many instances the tensile strength even exceeds the strength of plain 1:3 mortar, although, at the same time, the crushing strength of concrete mixed with the same kinds of oils is invariably somewhat less than that of plain mixtures. Only one oil, thus far, has been found to affect the tensile strength seriously, and this oil perhaps more nearly resembles the asphaltic California products than any of the others used.

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* Testing Engineer, Office of Public Roads, U. S. Dept. of Agriculture.

Mr. Goldbeck.

As has been stated, the influence of lubricating oils on plain concrete foundations is, in a great many instances, very severe. Animal and vegetable oils disintegrate concrete, and many of the lubricating oils contain animal and vegetable oil admixtures. This disintegrating action is particularly severe on lean, absorbent mixtures which may take up a very large quantity of oil. The mineral oil concrete mixture must be regarded in a different light from that of a plain concrete subjected to the penetrating action of lubricating oils. It is true that a very large percentage of oil, when mixed with the wet concrete, will cause it to lose all strength, but such excessive admixtures are unnecessary as well as dangerous. A small percentage, however, of the right kind of oil seems to be of benefit as a damp-proofing agent, and does not seem to be dangerously detrimental to the strength of the mixture.

Mr. Taylor.

ARTHUR TAYLOR, ESQ. (by letter).—It is very pleasing to note that Mr. Page has been able to supply some additional data on his oil-mixed concrete investigation; primarily, because it undoubtedly furnishes the incentive for the Profession to enter further into this comparatively new field of concrete experimentation.

It has been the desire of the writers, since their inextensive tests were undertaken, that they be instrumental in creating an interest in the investigation of this possibly economical means of water-proofing mortars and concrete, and they now believe that, due to Mr. Page's discussion, their hopes will be realized to some extent.

At the outset, the writers were quite confident that Mr. Page had demonstrated an economical and practical means of water-proofing mortars and concrete by the admixture of a mineral oil with wet mortars and concrete. In this belief, it was expected that they would be able to verify some of his conclusions and enlarge on the scope by the use of Western (asphaltic base) oils.

Somewhat negative results were first observed when the strength of the 7-day tensile specimens fell off with the "oil-mixes" to a greater relative degree than in Mr. Page's tests. In no case did the strength of the oil-mixed briquettes approach that of the plain mortar; nor did the strength of the oil-mixed specimens reach 80% of that of the no-oil specimens in the 28-day tests. Again, with the permeability test, the first specimens were watched to see the effect of the oil, and it was observed with regret that the oil-mixed specimens were more permeable than the plain ones, for it was hoped that results similar to Mr. Page's would be obtained.

Regarding the absorption tests, the noticeably slight absorption may be accounted for by the fact that the specimens were the broken halves of briquettes, undoubtedly poor examples from which to show normal absorption, but reasonable indicators of relative absorption of the various oil-mixed mortars. It was the practice of the writers

Mr. Taylor.

to mix the mortars with exceptional thoroughness, and to pack the moulds quite vigorously, so that, without doubt, the latent voids were greatly reduced. This naturally brought about low absorption.

All specimens were taken out of the water at the same time, wiped, "baked" in an open gas oven at 160° Fahr., weighed, and again placed in water. At the end of the 24 and 72-hour periods the water was wiped off and the specimens were again weighed.

In the permeability test, though the age of the specimens was not great, the results of the plain concretes and the various oil-mixed ones, with relation to each other, should be quite correct. It is likely that the oil ingredient retards the time required for the concrete to reach its normal state, or permanent set; however, the writers cannot help but believe that the results of long-time tests would follow lines similar to those at 28 and 35 days, but probably with less divergence between the no-oil and oil-mixed specimens. By an oversight, the age of the 4-day test was omitted. These tests were begun on the thirty-fifth day.

The writers wish to make clear that the reporting of their tests was no flaunting declaration that oil as an incorporated ingredient in concrete is not an aid to its water-proofing qualities. It would be lamentable, indeed, if experimenters lacked the courage to make public their findings, merely because they did not agree with those of others. Believing in this principle, they offered their conclusions.

Though Mr. Page does not claim that oil adds to the strength, nor does the extra manipulation required to incorporate the oil (for it is not so shown in his compressive tests), yet the writers fail to comprehend why so many of Mr. Page's oil specimens have shown greater strength than the plain mortars. They have been inclined to believe that the "personal equation" is a factor in the results of new-field experimental work of this nature, regardless of how conscientiously it is carried on, and they dare say that it is likely that a repetition of many of these tests, where equative handling is paramount, would show appreciable reversals.

To some extent, the applicability of the colloidal theory does not seem futile. Considering oil the colloid, "that amorphous part of 'clays' which envelops the crystalline constituent, * * * when it is in excess, the clay is most plastic, and when it is dry, it is the less permeable and less absorptive." This, however, is not in perfect analogy to cement, where a mechanical-chemical change takes place; therefore, can we impose the colloidal theory here?

Again, it is plausible to believe that the oils of different character account to some extent for the variation in the results obtained from the experiments by Mr. Page and those by the writers.

J. B. Lippincott, M. Am. Soc. C. E., reported* that the tensile

*Transactions, Am. Soc. C. E., Vol. LXXIV, p. 283.

Mr. Taylor.

tests in the laboratory of the Los Angeles Aqueduct showed lack of uniformity, and were not considered to be encouraging for the use of California oils with concrete. These tests, however, were for tensile strength only, and no comment on any others was made.

When we consider the extent to which oil-mixed mortars and concretes has been investigated, it is only infinitesimal compared with what has been done with plain mortars and concretes. How many dozens of men have brought to light new and diverse results in their studies, but every result showing the investigators new light, and enticing them to experiment further. Therefore, this should encourage men to undertake more detailed study. Whether or not the writers' work led to the correct conclusion, they rest satisfied that they have created a stimulus for further investigation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1266

ADDRESS
AT THE ANNUAL CONVENTION, IN OTTAWA, ONTARIO, JUNE 18TH, 1913.

SOME TENDENCIES AND PROBLEMS OF THE PRESENT DAY AND THE RELATION OF THE ENGINEER THERETO.

By George Fillmore Swain, President, Am. Soc. C. E.

In asking your attention this morning to the remarks which I shall offer for your consideration, I cannot forbear expressing to you, once again, my appreciation of the honor which you have done me in electing me to the position in which I find myself. It is a distinction which I shall always value above any other, and it involves a responsibility to the profession of which I am deeply sensible. This feeling, together with my conviction of the importance of our profession as a part of the social organism, has guided my choice of a subject for this address.

As we gather here, interested in widely divergent branches of engineering science, it has seemed to me that I could most profitably occupy the time by asking your attention to some matters in which we all should be equally interested. I have, therefore, chosen for my subject a consideration of "Some Tendencies and Problems of the Present Day," and the part which the engineer should play in their solution.

We are living in a most remarkable age, an age different from any that has preceded it in the history of the world, and one in which changes are taking place with marvelous rapidity. As a profession, we are largely responsible for the present conditions, and therefore we should

do our part in solving such problems as exist, and in helping to direct aright the tendencies of the day. We have a duty to ourselves, to our profession, to society, and to our successors, and we must perform it.

We have been often reminded of the fact that in one sense engineering is the oldest of the professions, since engineering constructions of some form are necessary even in the rudest communities; but in another, and perhaps a truer sense, it is the youngest of the professions. The priesthood is doubtless the oldest distinct profession recognized as such, for the most uncivilized tribes had priests to mediate between themselves and their crude deities. The priests were also lawgivers, practiced the healing art (such as it was), and, as civilization progressed, were also the builders. Bridges and temples were built by them, and the most stately buildings were those devoted to ecclesiastical uses. In course of time, the lawgiver, the physician, the architect, became distinct, and then for a time the engineer was either an architect or a priest. It is fair to say that engineering is the last profession to be differentiated into a class by itself, and that its beginning dates from about the Seventeenth Century, later receiving its great impetus from the series of inventions which were made toward the end of the Eighteenth Century—the spinning jenny, the loom, the puddling process, the steam engine, and the steam locomotive. Since that time, the profession has grown by leaps and bounds, has become separated into a dozen branches, and has probably become, directly or indirectly, the chief occupation of mankind. If this age may be designated as predominantly that of any profession, it is certainly the age of the engineer or applied scientist. He furnishes us with thousands of comforts, necessities, and luxuries, of which previous generations never dreamed, and which they would have laughed at as impossible. By supplying the fundamental material elements of modern civilization—transportation, the transmission and dissemination of intelligence (by telegraph, telephone, and printing press), machinery, prime movers, and the working of metals—the applied scientist becomes the minister to the other and older professions, furnishing tools for the surgeon and dentist, and chemical products for the physician; while engineering projects and problems probably supply as large a field as any for the employment of lawyers. The priest—originally lawgiver, physician, and builder—now stands alone, concerned only with the moral law and its application, and with our relation to the Infinite.

Civilization, as we know it to-day, then, seems to me to be due mainly to the engineer, or applied scientist, using the term in its widest sense. Of course, a great ethical advance has also been made, of which any student of history must be fully conscious; but, when we consider that moral principles were known and recognized by the ancients, in as perfect a form as that in which they can be stated to-day, without producing much, if any, wide-spread ethical progress, or any advance in civilization, for many centuries, we must, I think, conclude that it is the dissemination of intelligence, the facilities for transportation, the development of machinery, which have made the whole world kin, and thus have been the chief elements in promoting the universal brotherhood of man and the practical recognition of human rights, and so have been the main agents in the progress of civilization. This, somewhat differently expressed, is, as I understand it, the position taken by Buckle, often misunderstood by those who assume him to assert that no ethical progress has been made. However this may be, few will deny that the work of the engineer, if not the cause, has been a necessary condition, of progress.

This progress, both material and ethical, has been wide-spread, and astonishing in degree. In material things, not only necessities, but comforts and luxuries of which our forefathers would never have dreamed, are now within the reach of every man who is sober and industrious. Wages have risen, not only in absolute amount, but in purchasing power. The poor can now receive medical advice, medicines, and many other things free, where our predecessors could not have obtained the same things at all. Simple foods in great variety, and clothing of good quality, can be obtained at reasonable prices; and in every respect the poor man to-day is better provided for than his predecessors were. Ethically, too, he is far better off. The best books are available, if he wants them, at ridiculously low prices, free public schools are provided for his children, and all sorts of free industrial and vocational instruction is available for him in his leisure hours if he desires to use them. His hours of labor have been shortened, his civil rights have been generally recognized, he is treated as the equal of any man before the law, and his right to a fair chance in life—to opportunities commensurate with his ability to make use of them—is generally admitted in theory, if not yet entirely in fact. With free public and trade schools, with conditions which allow a common laborer

in a steel mill at 15 to become the head of the greatest industrial organization in the world at 50, certainly it cannot be said that men do not have opportunity in the United States.

Continued progress, however, and the interest of the social organism as a whole, require that individual initiative and ability shall be encouraged to the utmost and allowed to enjoy the reasonable fruits of its exercise; that property shall be protected; that taxation shall be equitable and uniform; that leaders shall be chosen from those most enlightened, capable, honest, and judicious; that those who are only fitted for manual labor shall not acquire a distaste for it or look down upon it as inferior in dignity to other occupations*; and that waste and extravagance shall be reduced to a minimum. There should be a recognition of the facts that wealth must be unequally divided, since men are unequal in ability and in character; that the prosperity of one depends on the prosperity of all;† that each man must feel secure in the enjoyment of all that he can legitimately win; that wealth, position, and luxury, do not in themselves bring happiness; and that the selfish desires of the individual must be subordinated to the interests of society as a whole if progress and not retrogression is to ensue. A state of equality of condition, as has been well remarked, would mean equality, not of wealth, but of wretchedness.

The political and social evolution in the past, up to a comparatively recent period, has not involved any diminution of the incentive to individual effort, nor to any great extent the attempt to deprive the individual of the fruits of his industry and ability. Of late years, however, with growing political power in the hands of the less intelligent classes, symptoms of a change have shown themselves. The spectacle of isolated instances of great wealth acquired unfairly or too rapidly, and of rewards out of proportion to service, instead of being looked upon as necessary phenomena, seeing that men are human, and that no human affairs can ever be perfect, has led to a wide-spread spirit of envy and discontent, and sometimes to a desire to deprive men of the results of their honest toil.

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*"But we exhort you brethren * * * that ye study to be quiet, and to do your own business, and to work with your hands," wrote the Apostle Paul.

†"This, then, above all, ought to be regarded by every one as an established principle, that the interest of each individual and that of the entire body of citizens are identical, which interest, if any one appropriate to himself alone, he does it to the sundering of all human intercourse." Cicero, "De Officiis."

It is to some unfavorable phenomena incident to this movement that I wish to call your attention to-day, believing that, if they continue, disaster may result, in which case, as always, the suffering will mainly come upon those least able to bear it.

1.—In the first place, let me refer to the tendency to consider men equal, or to what I may term a false democratic ideal.

What I conceive to be the true democratic ideal is this: that all men are equal before the law—so that neither differences in wealth, race, social position, nor talent, shall confer any advantage or impose any disadvantage in the impartial administration of justice; that while ability should be admired, every man who is honest, industrious, sober, and faithful, and who does his work in the world as well as he can and with due regard to the rights of others, is deserving of equal respect and regard, whether he occupy the humblest or the most exalted position; that success depends on the spirit with which one's natural endowments are utilized and the degree to which they are developed, and not on those endowments themselves, so that the humblest laborer on the street may be making as great a success of his life as the most gifted; that there should be kindliness and a spirit of brotherhood between all men, and neither envy of those more capable, who receive greater rewards, nor contempt of those less gifted, who do the menial work of the world; and that neither race, poverty, nor inferior social position, should prevent the possibility that a man, by hard work, integrity, and conscientiousness, should be able to reach, by some available ladder, a position commensurate with his abilities. A man should be judged by what he is, not by what he has, whether he is black or white, or who his grandmother was. He should have the opportunity to get such education as best fits his natural qualifications. Rich and poor should be judged by the same standard—inherent worth. But, though respect should be equal, rewards should not be. They should be in proportion to the value of the service, and governed by the law of supply and demand; nor are great rewards necessary for happiness.

Unfortunately, a democratic government, especially with universal suffrage, instead of tending to the realization of this ideal, tends in some respects directly away from it, and leads not seldom to the grossest perversions of it. The equality of man, which the framers of the Declaration of Independence held to be self-evident, instead of being

interpreted as above, is held to mean that men are inherently equal in all respects.

It is self-evident that, instead of being equal, men are very unequal. There is perhaps more difference between the most intelligent and the least intelligent voter than there is between the latter and an intelligent animal. It is held that all, being equal, should have equal opportunity; but why should there be equality of opportunity for unequal individuals? As Harris remarks, "Inequality of opportunity for unequal men is a nearer approach to true equality than equal opportunity for unequal men." Why, for instance, should all men have opportunity for a college education, when only a few can profit by it, and the great majority should work with their hands, not with their heads? Not every man who is able to pass successfully through a college course is able to use it when he has finished, and by its means many a good mechanic has been spoiled to make a poor and discontented clerk or lawyer.

One of the conditions of individual progress is the willingness to recognize and admire superiority, and without envy to rejoice in its success. The idea of the equality of man limits this recognition, and thus hampers individual progress, and therefore collective progress. In college, there is little in class associations to stimulate a man to excel. If he does, he is termed a "grind" and is looked down upon. If all men are equal, he has no right to exhibit any superiority to his fellows. The same is true among the so-called laboring classes. No man must do any more work, or do it any better, than another. Individual excellence and initiative are discouraged. Indeed, individual freedom is not seldom infringed upon, and a man is not allowed, at times, to work at wages satisfactory to himself, when he desires to do so—a strange result of freedom, and in reality as great an outrage as any that can be quoted from the mediæval annals of despotism. With growing discontent, many minds turn to socialism, that impracticable Utopia which, as President Butler has well said, "would wreck the world's efficiency for the purpose of redistributing the world's discontent."

The idea of the equality of man, instead of allowing the best men to govern and fitting the man to the proper vocation, leads to the placing of unfit men in power, the control of large masses of men by demagogues, the judging of men by the number of votes they can

control; and thus it leads to the withdrawal of many of the best men from active participation in public affairs, because they see how little influence they can exert without the use of money or the arts of the demagogue, and that the only reward of one who unselfishly serves the public is likely to be criticism and contumely. It also discourages loyal and faithful service, and makes difficult the maintenance of proper relations between employer and employee. Where every one is his own master, it follows that every one is his own servant.

What the ultimate result will be, cannot be foretold. The political doctrine of the equality of man is still on trial. Yet, as regards the United States, where that doctrine has its fullest expression, Lincoln long ago observed that if it fails, it will not be on account of interference from without, but it will be by suicide. It behooves us, therefore, if we can, to see to it that our institutions shall not fail by suicide.

Nevertheless, it is a somewhat striking fact that several of the most philosophical and learned historians have distrusted the ultimate success of our institutions. Froude, Macaulay, and Lecky, all express the gravest doubt of the permanence of our form of government.*

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*Froude, for example, writes, "I am no believer in democracy as a form of government which can be of long continuance. It proceeds on the hypothesis that every citizen is entitled to an equal voice in the management of his country, and individuals being infinitely unequal—bad and good, wise and unwise—and having rights dependent upon fitness to make use of them, the assumption is untrue, and no institution can endure which rests upon illusions."

Macaulay, too, had great doubt of the permanence of our form of government. One of his letters to a friend in America has been often quoted, but, for convenience in this connection, it may be well to reproduce it here.

"Holly Lodge,
Kensington,
LONDON, May 23, 1857.

"DEAR SIR:

"You are surprised to learn that I have not a high opinion of Mr. Jefferson, and I am surprised at your surprise. I am certain that I never wrote a line, and that I never in Parliament, in conversation, or even in the hustings—a place where it is the fashion to court the populace—uttered a word, indicating an opinion that the supreme authority in a State ought to be intrusted to the majority of citizens told by the head; in other words, to the poorest and most ignorant part of society. I have long been convinced that institutions purely democratic must, sooner or later, destroy liberty or civilization, or both. In Europe, where the population is dense, the effect of such institutions would be almost instantaneous. What happened lately in France is an example. In 1848 a pure democracy was established there. During a short time there was reason to expect a general spoliation, a national bankruptcy, a new partition of the soil, a maximum of prices, a ruinous load of taxation laid on the rich for the purpose of supporting the poor in idleness. Such a system would, in twenty years, have made France as poor and barbarous as the France of the Carlovingians. Happily, the danger was averted; and now there is a despotism, a silent tribune, an enslaved press. Liberty is gone, but civilization has been saved. I have not the smallest doubt that, if we had a purely democratic government here, the effect would be the same. Either the poor would plunder the rich, and civilization would perish, or order and prosperity would be saved by a strong military government, and liberty would perish.

"You may think that your country enjoys an exemption from these evils; I will frankly own to you that I am of a very different opinion. Your fate I believe to be certain, though it is deferred by a physical cause. As long as you have a boundless extent of fertile and unoccupied land, your laboring population will be far more at ease than the laboring population of the old world; and while that is the case the Jeffersonian polity may continue to exist without causing any fatal

Their predictions, we hope, will prove erroneous; we must make them so, if we can. Nevertheless, the more we reflect, the more I think we shall realize that out of the false theory of the equality of man have sprung many real dangers. A spirit of unhealthy discontent has been aroused, and the people, who are sovereign, endeavor, by various kinds of legislation, to realize in fact the untruth of equality. They make the common mistake of assuming that legislation is a sovereign remedy for all ills. Not knowing how to proceed, they become the prey of every selfish demagogue who, while stirring them up to exaggerate their troubles, and professing his ability to remedy them, is simply seeking his own ends. These men know that a large part of the voters to whom they must appeal are ignorant, easily swayed by claptrap, and deceived by hypocrisy, "incapable of disentangling a difficult question, judging distant and obscure consequences, realizing conditions of thought and life widely different from their own, estimating political measures according to their true proportionate value, and weighing nicely balanced arguments in a judicial spirit." The

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calamity. But the time will come when New England will be as thickly settled as Old England. Wages will be as low, and will fluctuate as much with you as with us. You will have your Manchesters and Birminghams; and in those Manchesters and Birminghams hundreds of thousands of artisans will assuredly sometimes be out of work. Then your institutions will be fairly brought to the test. Distress everywhere makes the laborer mutinous and discontented, and inclines him to listen with eagerness to agitators who tell him that it is a monstrous iniquity that one man should have a million while another cannot get a full meal.

"In bad years there is plenty of grumbling here, and sometimes a little rioting; but it matters little, for here the sufferers are not the rulers. The supreme power is in the hands of a class, numerous indeed, but select—of an educated class—of a class which is, and knows itself to be, deeply interested in the security of property and the maintenance of order. Accordingly the malcontents are gently but firmly restrained. The bad time is got over without robbing the wealthy to relieve the indigent. The springs of national prosperity soon begin to flow again; work is plentiful, wages rise, and all is tranquility and cheerfulness. I have seen England pass, three or four times, through such critical seasons as I have described. Through such seasons the United States will have to pass in the course of the next century, if not of this. How will you pass through them? I heartily wish you good deliverance. But my reason and my wishes are at war, and I cannot help foreboding the worst. It is quite plain that your Government will never be able to restrain a distressed and discontented majority. For, with you, the majority is the government, and has the rich, who are always the minority, absolutely at its mercy. The day will come when, in the State of New York, a multitude of people, none of whom has had more than half a breakfast, or expects to have more than half a dinner, will choose a Legislature. Is it possible to doubt what sort of a Legislature will be chosen? On one side is a statesman preaching patience, respect for vested rights, strict observance of public faith; on the other is a demagogue, ranting about the tyranny of the capitalists and usurers, and asking why anybody should be permitted to drink champagne and to ride in a carriage while thousands of honest folk are in want of necessaries. Which of the two candidates is likely to be preferred by a workman who hears his children cry for bread?

"I seriously apprehend that you will, in some such season of adversity as I have described, do things that will prevent prosperity from returning; that you will act like people who should, in a season of scarcity, devour all the seed corn, and thus make the next year not one of scarcity, but of absolute famine. There will be, I fear, spoliation. The spoliation will increase the distress. The distress will produce fresh spoliation. There is nothing to stop you. Your Constitution is all sail and no anchor. As I said before, when a society has entered on its downward progress, either civilization or liberty must perish. Either some Caesar or Napoleon will seize the reins of government with a strong hand; or your republic

demagogue, therefore, appeals to class interests and class animosities, and does his utmost to foment a spirit of discontent, even though every condition justifies the opposite, gaining his reward by newspaper headlines, notoriety, public office, or not seldom, wealth secured at the expense of the public that he deceives. The spirit of servility and sycophancy which in former times led men to grovel in the dust at the feet of kings and princes, is still with us, unchanged in form, only its object is different, and its adulation is now directed to the sovereign voter—the more ignorant, the more easy to flatter and deceive. Knowing human nature, may we not doubt the man who claims to act for the good of the people; and trust him rather who frankly admits that his actions are governed by his selfish interests, but who has learned that these can only be served by promoting the best interests of all.*

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will be as fearfully plundered and laid waste by barbarians in the twentieth century as the Roman Empire was in the fifth—with this difference, that the Huns and Vandals who ravaged the Roman Empire came from without, while your Huns and Vandals will have been engendered within your own country by your own institutions.

"Thinking thus, of course I cannot reckon Jefferson among the benefactors of mankind.

"Yours respectfully,
"THOMAS BABINGTON MACAULAY."

Lecky, one of the most thoughtful of English historians, expresses in a number of places his doubts regarding the success of democracy. For instance, he says:

"Strong arguments may be adduced both from history and from the nature of things to show that democracy may often prove the direct opposite of liberty. * * * Democracy destroys the balance of opinions, interests, and classes, on which constitutional liberty mainly depends, and its constant tendency is to impair the efficiency and authority of parliaments, which have hitherto proved the chief organs of political liberty. * * *

"A despotism which secures order, property and industry, which leaves the liberty of religion and of private life unimpaired, and which enables quiet and industrious men to pass through life untroubled and unmolested, will always appear to many very preferable to a democratic republic which is constantly menacing, disturbing or plundering them." (Lecky, "Democracy and Liberty," pp. 256-260.)

Again, he says, in another place:

"In every field of human enterprise, in all the competitions of life, by the inexorable law of Nature, superiority lies with the few and not with the many, and success can only be attained by placing the guiding and controlling power mainly in their hands. * * * Yet, surely, nothing in ancient alchemy was more irrational than the notion that increased ignorance in the elective body will be converted into increased capacity for good government in the representative body; that the best way to improve the world and secure rational progress is to place government more and more under the control of the least enlightened classes. The day will come when it will appear one of the strangest facts in the history of human folly that such a theory was regarded as liberal and progressive." (*Ibid*, p. 26. Speaking about parliamentary government, he says: "In all the instances in which this form of government has been conspicuously successful, the representative body was returned on a restricted suffrage." The great German historian, Sybel, said: "The realization of universal suffrage and its consequences has always been the beginning of the end of all parliamentarism." Professor Freeman said: "The real special weakness of pure democracy is that it almost seems to require slavery as a necessary condition of its existence." Aristotle, Goethe, and Voltaire, all disbelieved in a purely democratic form of government.)

*"The men who, in former ages, would have sought by Byzantine flattery to win power through the favor of an emperor or a prince, will now be found declaiming on platforms about the iniquity of privilege, extolling the matchless wisdom and nobility of the masses, systematically trying to excite their passions or their jealousies, and to win them by bribes and flatteries to their side." Lecky, "Democracy and Liberty," p. 30.

Some of the remedies for these dangers, it seems to me, are obvious. In the first place, we should realize that the statement that all men are created equal, properly interpreted, simply means that they are equal before the law, that all men should be treated with justice, should have fair opportunities, and that each should be secured in his right to the products of his own labor, and to his own liberty, so far as his acts do not injure others. Aside from this, instead of considering men equal, we should recognize and encourage inequality, for it is easily to be seen that the progress of society depends on it. Equality of condition means pauperism or savagery; the inequality of man means the division of labor, progress, civilization.

In the second place, we should encourage the recognition and admiration of superiority.

In the third place, we should preach the gospel of content, instead of discontent. "Do violence to no man, neither exact anything wrongfully; and be content with your wages" said John the Baptist. There are two kinds of discontent: one praiseworthy; one ignoble. The former springs from a laudable ambition and a desire to perfect oneself, so far as natural endowment will permit. The other springs from envy and the desire to reap the rewards of the industry of others. The former kind of discontent is to be encouraged; every man should be given an opportunity to develop himself and to be confirmed in the possession of the prizes which he may gain; but the discontent which springs from envy, which leads men to depend on government help, instead of self-help, which sanctions forms of unjust taxation which may be nothing less than legalized robbery, should in every possible way be repressed. The attention of men, instead of being concentrated on what they want, should be directed to a realization of what they have. In contrast with the conditions which existed one or two centuries ago, even the conditions of the poorest classes at the present time are immensely improved. Yet with all the reduction of working hours that has taken place in the last 50 years, and the other improvements in condition, I doubt if there is any more real happiness and content among the poorer classes. Nor have I any doubt that the excessive talk about bettering the condition of the working classes blinds them to the opportunities for their own thrift and industry.

Unfortunately, it is a vain hope that there will ever be an absence,

or even a dearth, of demagogues to stir up this spirit of unworthy discontent, but we should do all in our power to counteract it. Much of it is due to the unwise use of wealth by those who possess it, who think that money can buy everything, and is the end and aim of existence, and who, instead of being governed by a feeling of the brotherhood of man, and the spirit of respect for true worth, even in the humblest occupations, assume an attitude of superiority, or even of insolence, toward those less fortunate than themselves.

It is also vain to hope that we can ever reach a point where there will be no poverty.* This would realize what Dr. Johnson termed the triumph of hope over experience. It would mean a radical change in human nature. As long as there are masses of men who are shiftless, lazy, incompetent, and vicious, whom nobody would willingly employ, poverty will exist. Still, every effort should be made to relieve it where it is undeserved. It will be well, however, to bear in mind that suffering is a remedy, and that unwise philanthropy may do harm instead of good. If men will learn in no other way the lessons of life, they must learn them by suffering, which is Nature's cure. Of course, it is often hard to distinguish deserved from undeserved suffering. The latter may be wisely relieved; the former generally not. Surely an all-wise Creator would not have filled the world so full of it except for some good purpose, and that purpose cannot be to afford to others the opportunity for relieving it, for much of it cannot be relieved.

2.—The Tendency to Disregard Authority.—Out of the tendency to look upon all men as equal, spring some other tendencies to which I will refer. One of them is to disregard authority, and to consider that one man's opinion is as good as that of any other. We see this tendency about us every day; it leads to intellectual arrogance, dogmatism, and lawlessness.

Every man has presented to him, almost daily, questions which he is not competent to decide for himself, either because he has not had the necessary time, training, or experience, or because he is incapable of judging.

*"Its causes lie chiefly in incompetence, lack of energy, bad heredity, and unhealthy surroundings, rather than in a vicious economic system, or in lack of opportunity. The relief of poverty has become so judicious that few poor persons are left to suffer from want. The prevention of poverty is to be found in good sanitation enforced by the municipality, in suitable education of individuals according to capacity, and in self-help, rather than in economic revolution or in indiscriminate equality of opportunity." Harris, "Inequality and Progress."

In these days, when literature is so superabundant, it is most im-
rtant to choose our advisers wisely, to know whom to trust, to dis-
rn wisdom, to select the proper books to read, to choose the best
thorities to whom we shall listen, and, when we have so chosen, to
nsider attentively the message they bring, and to distinguish advice
at springs from self-interest from that which is disinterested. There
a great temptation to form hasty or immature judgments on slight
thority; and the habit, once formed, may be difficult to eradicate.

The disregard of authority is peculiarly observable in the younger
neration. It is not confined to the uneducated, but is even more
servable in many whose self-esteem has been increased by a course
college. Finding that men disagree on almost every subject, and
at even those in positions of authority do not hold the same opinion,
is easy to conclude that one man's opinion is as good as another's.
et us examine this matter further.

The subjects of study may be broadly divided into three classes:
irst, the relations of abstract ideas, that is, mathematics and logic;
cond, natural science; third, the so-called humanities.

With regard to subjects in the first class, no observation or ex-
rience is necessary, and there will be no differences of opinion. They
e not, properly speaking, matters of opinion at all. There can be
discussion as to whether two and two make four, or as to whether
e three angles of a triangle amount together to two right angles.

With regard to natural science, conclusions depend to some extent
observation, experiment, and experience. Hypotheses may be
rmulated, experiments made, conclusions deduced therefrom; and
ffering conclusions may be arrived at by different persons. In this
ass then, there may reasonably be divergencies of opinion. These
bjects, therefore, are less definite, more hypothetical, more uncer-
in, than those in the first class.

Under the third class are comprised what are called mental and
oral philosophy, politics, government, history, language, etc. This
lass of subjects is even more uncertain and less definite than the
cond. Experiments are less conclusive, and cannot be made under
terminate conditions, as in natural science, the data are more shift-
g and variable, experience is of great value, and extreme differences
opinion will often occur.

The thoughtful man will recognize these differences between the

various subjects of investigation. He will form his conclusions o subjects in the first class with definiteness and certainty, and wil be justified in feeling confident of their correctness. He will recogniz the greater indefiniteness of subjects in the second class, and if h forms definite opinions, he will hold them in a sort of tentative man ner, realizing that the next experiment or discovery, or greater ex perience, may show them to be incorrect. He will realize the sti greater indefiniteness of subjects in the third class, and though h may well form definite opinions regarding these also, he will hol them with still greater modesty, realizing that in almost every ques tion of this character there is much to be said on the other side, an that he may be entirely wrong. Only the man who has studied th subjects in all three of these classes will realize fully the differen degrees of certainty attaching to each. Such a man will be muc more modest and safe in his conclusions than one who has studie only one class of subjects. It may further be remarked that a ma who has studied only the third class of subjects, will be likely to b more dogmatic and less safe than one who has studied the first o the second, or both; because, not being familiar with subjects in whic a great degree of certainty, or even absolute certainty, is attainabl he may assume that such degree of certainty is attainable in the sul jects of the third class; whereas, a man who has studied mathematic or science, and is accustomed to definiteness, will, when he is confronte with problems of politics, economics, government, or ethics, be a once struck with the great difference in reliability of the data, the impo sibility in many cases of making crucial experiments, and the fluctua ing conditions involved. For this reason, it is not uncommon th scientists, mathematicians, or engineers hold and express opinion regarding politics, economics, or ethics, with much less dogmatism tha less well-informed men who have devoted their attention more strictl to these last-named subjects.*

*Sir George Cornewall Lewis, one of the deepest political thinkers of Englan or of all time, lays down the following rules regarding the qualities which rend a person a competent authority in matters of opinion:

"The first qualification is, that a person should have devoted much study an thought to the subject-matter if it be merely speculative; and that if it be practica he should also have had adequate experience respecting it.

"Secondly, his mental powers must be equal to the task of comprehending th subject, and they must be of the sort fitted to it.

"Thirdly, he ought to be exempt, as far as possible, from personal interest i the matter; or, if he be not exempt, his honesty and integrity ought to be such a to afford a reasonable security against the perversion of his opinions by views o his individual advantage." "On the Influence of Authority in Matters of Opinion. London, 1849, p. 27.

Lord Brougham, in his "Political Philosophy" (Vol. 1, p. 3), says:

"Mathematicians who run hardly any risk of error—naturalists who n but little more—have never been so bigoted and so uncharitable those whose speculations are fated to be always involved in more less of doubt; and when we come to political reasoners, we find, besides e intolerance of metaphysicians, a new source of error and of fault the excitement which men's interests, real or supposed, lend to their ssions."

Excellent books on economics have been written by mathematicians astronomers, and it is remarkable to observe how many works on ese subjects have come from men who have been senior wranglers Cambridge.

As Lewis says:

"The qualities which render a man a trustworthy authority in atters of opinion, are much rarer than those which render a man credible witness in matters of fact. Accordingly, the honesty which duces a man to speak the truth is more common than that which duces him to form sound opinions. There are many men who, der ordinary circumstances, would never be seduced by interest to port a fact falsely or to express an insincere opinion, whose judgnt might, nevertheless, be perverted by interest."*

Lecky also expresses the same thought when he says:

"There is such a thing as an honest man with a dishonest mind. ere are men who are wholly incapable of willful, deliberate unthfulness, but who have the habit of quibbling with their contions and by skillful casuistry, persuading themselves that what ey wish is right."

Indeed, one of the first principles which we must thoroughly grasp, we wish to judge correctly as to the authority of a writer, is that rsonal honesty and good intentions do not in themselves constitute y sufficient guaranty of authority. History, as well as the experice of almost every man, abounds in instances of the truth that the st honest and well-intentioned men may be the worst advisers. The ench have a witty saying that "virtue is more dangerous than vice, cause it is not subject to the restraints of conscience." Similar is e old saying, quoted by Cicero, that "The extreme of right is the treme of wrong."

Two of the most important requisites for authority in matters opinion, seem to be a spirit of conservatism, and the possession of

Loc. cit., p. 35.

good judgment. The man who is slow to advocate a change, who adheres instinctively to old ideas and practices until there is reasonable ground for the belief that a change will be beneficial, is clearly a safer guide than he who rushes into all sorts of new schemes and is ever ready to throw aside the results of years, because perfection has not been attained. Conservatism is the brake which prevents our rushing to destruction. The possession of grasp and of good judgment is also essential. By these are meant the ability to see all the elements entering into a problem and to see them in their proper proportion. The narrow-minded man does not see all the elements in a problem; such a man lacks grasp. The unbalanced man, though he may see all the elements, exaggerates some and minimizes others; such a man has poor judgment. The advice of either is unreliable.

It is worthy of fuller inquiry what the reasons are, reduced to their elements, which cause men to differ in their opinions. Take almost any subject, except one dealing with abstract relations, and opinions may be cited which are diametrically opposite. Consider such a matter, for instance, as the relation of forests to stream flow. Here we have a scientific subject, regarding which it might be supposed that experiments or observations might be made, scientific principles applied and opinions reached which would be agreed upon by all intelligent men conversant with the subject; and yet, on this subject, and still more so on even more speculative matters, men of equal authority or experience will be found holding diametrically opposite views. Why is it?

It seems to me that, if intelligent men differ on any subject, it must be due to one or more of the following causes:

a.—One or the other is not in possession of all the facts or principles involved;

b.—One or the other may use facts or principles which are incorrect;

c.—One or the other may reason incorrectly;

d.—One or the other may see the facts out of proportion, attributing too great importance to some and too little to others;

e.—One or the other may illustrate the natural stubbornness or imperviousness of the human mind.

It is not always clearly recognized that often facts cannot be compared directly, because they cannot be expressed except in incom-

mensurable terms. For instance, if we ask which is the greater, six feet or two meters, there will be no disagreement in the answer; but, if we ask which is the more important, 10 feet or $5, or which is more important, truthfulness or beefsteak, the reply is not entirely obvious. These are not fanciful illustrations. We are continually obliged to make comparisons between things which have to do with our physical well-being and those which have to do with our moral or spiritual well-being. They are incommensurable, yet they must be compared and an opinion formed from the comparison. In planning a railroad station, for instance, one design may require the passengers to walk a certain number of feet, while another design costs more but requires less walking. Here is a comparison which must be made between feet and dollars. In such a case, some will endeavor to express the comparison in commensurable terms. They will compute, perhaps, the value of the shoe leather that would be saved in a year if all the passengers using the station have to walk the given distance less, and compare it with the interest on the additional expenditure. Similarly, the expenditure of millions of dollars is asked for electrification, the advantage urged being the saving of a certain number of minutes each day for each passenger, and because the aggregate saving of time in a year, valued at the average annual wage of an individual, amounts to a sum greater than the interest on the cost. Such comparisons are generally made by those who find their judgment unable to deal with the subject in any other way—who cannot compare except by measuring or weighing, as they have been trained or accustomed to do. Yet such persons do not realize that the comparisons they make may be entirely deceptive. Although more rapid transit may render a journey quicker, it does not follow that the time is saved. It may be usefully employed in reading, or in conversation, or in sleep, or in meditation. An additional fifteen minutes spent in going from one's house to one's office, is not necessarily lost; it may be utilized like any other fifteen minutes in the day. Even if spent solely in meditation, it would be most beneficial, for, as Arnold Bennett says, the great defect of the age is the absence of meditation. In such arguments as that for electrification the mistake is also made of not considering whether the individuals who would make the saving claimed are the same individuals who would bear the increased cost.

Referring to the classification above, it is evident that an ability

to observe correctly, the faculty of getting at all the facts, and of knowing when all the pertinent facts have been collected, the ability to reason from those facts, the possession of conservatism, the absence of stubborn persistence in holding a previously formed opinion in the face of proof of its error, and, above all, the faculty of seeing things in proper proportion, or good judgment, are essential in forming an authoritative opinion.

How can the absence of these qualities be detected? To discuss this would lead us too far, but there are one or two points to which I wish to refer. For one thing, dogmatism, or intellectual arrogance in speculative or uncertain matters, is an almost certain indication of disqualification. The man who in such matters shows intolerance of opinion, may generally be disregarded. He who in matters of opinion is cock-sure, who terms all men who disagree with him knaves or fools, who resorts to epithets and extreme statements, who asserts that all honest men should believe as he does, or who makes similar assertions, is either willfully trying to mislead, or is unworthy of credence. This is a fault which is observable particularly in the immature, whether young or old in years.*

*Let me quote once more from Sir George Cornewall Lewis on this subject:

"Whenever a person, having formed an opinion upon grounds which appear satisfactory to himself, asserts it confidently, and adheres to it resolutely, without showing due deference to the authority of others, he is justly exposed to the charge of *arrogance or presumption* in judging. The opinion of experienced men, having a special acquaintance with the subject, is always entitled to weight, even if it be unsupported by argument. Hence, all young persons, who attempt to judge for themselves, either are, or appear to be, arrogant; as they can scarcely fail to set at nought the opinions of persons more experienced and of greater authority than themselves. It is difficult for a young man, on account of the narrow circle of his experience, to discriminate between the cases in which he ought to judge for himself, and those in which he ought to defer to the opinions of others. He ought not to form, in youth, a habit of blind submission to authority, such as the Jesuits inculcated upon their disciples; but a spirit of docility, and respect for the opinions of their elders, undoubtedly becomes the young; and it is certain that their opinions, especially on practical questions, however formed, will in general be of very little value to others, however important it may be to themselves to cultivate a habit of independent and conscientious judgment.

"Persons who have formed habits of independent thought and examination likewise generally subject themselves to the same reproach—inasmuch as they often attach an undue weight to a chain of reasoning which they have gone through in their own minds, as compared with the opinions of persons who appear to be entitled, by their experience, to pronounce authoritatively on the subject. It is no easy matter to define the point where firmness, resolution, and proper self-reliance begin. It is difficult, again, to discriminate between teachableness, humility, and reverence for high authorities, and a tame and passive submission of the understanding.

"Universally, indeed, it may be said, that no person ought to express a confident opinion upon speculative grounds, until he has ascertained, by careful investigation, the unsoundness of the reasons for a different opinion, held by practical and experienced men. If, however, after a due examination, he differs from their conclusions, he cannot, even if he be wrong, justly be taxed with arrogance. To persist in error after proper inquiry and reflection, is not presumptuous, although it may imply *other* moral or intellectual defects. * * *

"'That man' (says Hesiod, in some celebrated verses, which acquired in antiquity almost an oracular authority,)—'that man is the most excellent, who can always think for himself. He, too, is a good man who will take sound advice from others. But he who can neither think for himself, nor will listen to the sound advice of others, is a worthless man.'" *Loc cit.*, pp. 113-117.

If, therefore, we find an author proposing changes in political or social conditions who refers to eminent men at least as honest and intelligent as himself, and more learned and experienced than he is, in terms of disrespect, we may be pardoned if we surmise that his own temper and judgment are not such as to entitle his words to be received with authority. So also, if we detect what we believe to be frequent gross errors in reasoning, or a temperamental lack of the power to see things in what is clearly their proper perspective, we may save ourselves the trouble of pursuing such an author farther, even though his sentences are adorned with flowers of rhetoric or by Biblical quotations.

3.—The Disregard for Experience.—One of the most serious tendencies of the present day, particularly in the United States, a tendency which, like the disregard of authority, seems to result from that to regard men as equal, is the disregard for experience. It is a tendency fraught with great possibilities for evil, which must be recognized by every serious thinker.

In any occupation, except those which have to do with the study of purely abstract relations, which, as above explained, may be arrived at correctly in the seclusion of the study, it is almost self-evident that experience is of great value. In any occupation having to do with pure or applied science, and particularly in those having to do with man and his relations to society—that is to say, in politics, economics and government, as well as in engineering—experience is a necessary qualification for arriving at correct judgments. A person having a power of perception of mathematical relations may become a profound mathematician at an early age; indeed, the perception of mathematical truths is essentially a subjective faculty, and may be most correct and powerful when the mind is removed from the objective state. Some mathematical prodigies have reached their greatest power when mere boys, and have lost it with increasing age and contact with the objective world. In business affairs, however, no amount of reading of books or of closet meditation can take the place of actual experience; but, if all men are equal, then the boy's opinion is as good as the man's, and experience is a mere useless drag.

It seems to be an inherent tendency of our form of government to disregard the value of experience; any man is considered good for any job; all he has to do is to get the votes, by one means or another. Men

who have made little or no study of government, and who have had no experience in it, are considered capable of judging of and administering laws, and of occupying any elective or appointive position. The butcher, the baker, the candlestick-maker, are all eligible for the state legislature, for congress, for the senate, for the mayoralty, for the presidency. This condition necessarily results in gross inefficiency and waste, not always through evil intent, but perhaps more often through stubbornness, ignorance, conceit, or the unwillingness or inability to select and take good advice. We talk much of graft, and rightly condemn it; but, as Lecky has pointed out, corrupt governments are not necessarily extravagant. Graft is injurious mainly to those who practice it. Inefficiency and inexperience are much worse for society in general; and laws which hamper industry, which harass property, or which—through unwise taxation, or extravagant state expenditures for the benefit of particular classes—plunder large portions of the community, are much more injurious to the general good than the pilfering of a few thousands of dollars a year. Sir Henry Maine remarks that the form of bribery which is most to be feared in a democracy is that of "legislating away the property of one class, and transferring it to another."*

Only in one department do we recognize experience, knowledge, and specialized training as necessary, and that department is the administration of justice. No one but a trained lawyer is considered eligible for the position of judge, and in most instances, judges are appointed and not elected, and are, therefore, not driven to seek the votes of the mob, or subject to popular whim, or the arts of the demagogue. For this reason, our Courts have been rightly deemed the safeguard of our liberties. Not until our legislators and our administrators of public affairs are chosen from men trained and experienced, and who have ascended from the bottom step by step, will efficiency, economy, and wisdom be attained in our government. Whether this time will ever be reached may well be doubted. We are much less advanced in this respect than some of the countries in Europe, where the expert is more recognized and has a higher standing, and where knowledge, and particularly experience, are valued more highly than here. If financial measures are under consideration, men experienced in finance should be sought and their advice followed. In our country, experience in finance

*"Popular Government," p. 106.

seems to be to-day looked upon as a disqualification for giving sound financial advice; and it is the same with railway and other affairs. Under such conditions, satisfactory results cannot be attained. Either, as a people, we are more dishonest, or more unwise, than the people of Europe.

In these days, when higher education is a fad, there is an increasing tendency, in my opinion very pernicious, to regard book knowledge as the equivalent of experience. We are more and more inclined to regard those as able to speak with authority, whose knowledge is derived simply from books, who perhaps have spent their lives in a professorial chair, with no actual experience in the subjects they teach or write about. We shall in time learn the fallacy and danger of this; but, in my opinion, at the present time great harm is being done by taking too seriously, in questions of politics, government, economics, finance, and business, and sometimes in engineering, the opinions of theoretical men and closet reformers, whose advice we would not think of following in the conduct of our own private affairs. Some of these men will come within the class of those who, to use the phrase of Mr. E. P. Ripley, President of the Atchison, Topeka and Santa Fé Railroad, have "zeal without knowledge, and enthusiasm without sanity."

Of course, it must not be supposed that experience is everything. In some cases, experience only confirms a man in rule-of-thumb practice, works him deeper and deeper into a rut, and destroys his power of vision. Probably all of us have seen instances of men, whose long experience in doing a particular thing in a particular way has only destroyed their power to see that there is another and a better way, and whose only reason for doing that thing in that way is that they have always done it in that way. Such men, however, would never, under any circumstances, be leaders. They are the soldiers in the ranks, whose only function should be to obey orders. It not infrequently happens, in the administration of some business affair, that a new man with a fresh mind, without experience in that line, but with a power of selecting good advisers, of grasping a new situation quickly, of seeing all the elements involved and of judging them correctly, will arrive at a truer decision than another man of less grasp and judgment, though experienced in the matter in question. This, however, is no argument against experience, but simply illustrates that experience must be combined with grasp and judgment; for the new

man, before he becomes really proficient, must either gain his experience, or must be able to select his advisers well and to rely on them. If we are obliged to choose between judgment and experience, we may well prefer judgment; but experience added to judgment is unquestionably better than judgment alone.

Not only is experience discredited to-day, but we actually find inexperience put forward as a virtue by men who should know better. One candidate for an important elective office, in an address to the electors just previous to the election, urged his qualification for the post on the ground that he had had no business experience which would prejudice his judgment in the matters which would come before him. Is not this a serious disqualification—an admission that his temperament is such that experience would prejudice him, and not teach him? Do we not want men in office experienced in the affairs they are to deal with, but fair-minded and of good judgment, whose experience, instead of warping their judgment, aids and matures it? Woe be to us as a nation when we place inexperienced men in office because experience would warp their judgment.

It is, nevertheless, true that men differ greatly in the amount of experience which they need in order to perfect their powers. This will depend on the rapidity with which they can assimilate it, and the philosophical character of their minds. It has been said that the wisest man is he who can do with the least experience; that is to say, the one who does not need to have an experience repeated again and again before he learns the lesson which it teaches, but who only needs one experience to learn its lesson, and who can extend that lesson to cover other possible experiences, thus rendering the latter unnecessary.

When we are asked, therefore, to believe and advocate any proposed policies, let us train ourselves to apply certain touchstones before we commit ourselves. If the subject is one within our experience, knowledge, and reasoning powers, let us endeavor earnestly to think the matter out for ourselves and arrive at our own conclusions. In the many cases in which we cannot do this let us carefully scrutinize the policies and their proposers, to determine how much credence we can give. Let us observe whether the author asserts them dogmatically or modestly; let us note whether he makes assertions without proof, and by incessant repetition endeavors to make us believe that assertion constitutes demonstration; let us ask whether he speaks from

experience or simply from book knowledge; let us remember that he is not free from the frailties of human nature, and let us, therefore, inquire whether he has any personal interest at stake; let us assume an attitude of doubt and distrust toward all human opinions until that doubt and distrust are effectually removed. It is said that it is better to be deceived than to distrust; but, after we have been deceived, we shall be more inclined to reverse this saying, and to consider that, in the reversed form, it will be, upon the whole, safer and more conducive to our happiness. Let us be on our guard against assuming that good intentions are a guarantee of authority, and when we find ourselves in danger of being influenced by flowers of rhetoric, let us remember that "no man is more dangerous in a state than he who possesses in an eminent degree the power of moving, dazzling, and fascinating his contemporaries, while in soundness of judgment he ranks considerably below the average of educated men"; and let us further bear in mind that "an excessive love and admiration of rhetoric is one of the diseases to which democratic communities are most liable."

4.—The Tendency to Relax Discipline.—Another of the demoralizing tendencies of the equality of man is that which has led in recent years, it seems to me, to the gradual but steady relaxation of discipline. It will probably be denied by few that for the proper training and full development of the human being, discipline is necessary. Life itself is little more than discipline; it is not for pleasure; it is for work, accomplishment, development; without discipline there will be no proper development. He who is born with a golden spoon in his mouth and whose life is a continual round of pleasure, is likely to go out of the world less developed than when he entered it. "The beginning of wisdom," says Solomon, "is the desire of discipline." Work, which is itself an end, and rather to be sought than what it brings, consists largely in doing things which in themselves are uninteresting and perhaps distasteful, and we must accustom and train ourselves to be able to do this kind of work cheerfully and well. This ability will be attained only through discipline. Moreover, the man who attains a position of responsibility must be able to direct and command the work of others, and no one can command wisely until he has first learned how to obey. For this, too, discipline is necessary. Yet the tendency of recent years seems to me to have been toward a steady

relaxation of the discipline imposed by law and custom upon the growing individual.

A tendency to relax discipline is strikingly shown where its necessity is obvious, namely, in our educational system. Formerly, the boy was put through a rigid course of prescribed study, some of it interesting and some uninteresting to him; but all of it, presumably, if the course was properly prepared by a capable faculty, necessary for the end in view. The modern tendency has been to substitute what he chooses to do or likes to do. The result has been, and is, that many young men come out of our colleges with minds and faculties practically wild and undisciplined, without willingness to do as they are told; and without the faculty of concentrating their attention on unpleasant but necessary tasks; but, if all men are equal, why should the teacher have the power to impose unpleasant tasks upon the pupil?

That the tendency in education has been to relax discipline, will not, I think, be questioned by many educators who themselves appreciate what discipline is. I understand, for instance, that the National Educational Association has recently said that high schools should be allowed to omit the study of algebra and geometry, and that the colleges should be compelled to accept for admission an equivalent amount of "science." As if we could study science intelligently without a knowledge of elementary mathematics! President Hadley, in a recent address,* comments upon this and emphasizes the fact that real training in science depends upon the method, not the subject; and he adds that "fifty years ago the one course in the academic department of Yale College where modern science was really taught, was the course in freshman Greek," because it was taught by the scientific method. Would that we had more teachers like the Hadleys, father and son. Our teachers are generally, I think, chosen upon an incorrect principle; they are appointed by reason of what they know; it seems to me they should be selected for what they are—for their ability to teach, and their power of enforcing scientific discipline. We may well ponder to-day the words of that fine old English schoolmaster, Richard Mulcaster, who died 302 years ago, and who wrote:

"I would rather hazard the reproach of being a severe master in making a boy learn what may afterwards be of service to him, even though he be negligent and unwilling at the time, than that he should

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lack any advantage when he is older, because I failed to make him learn, owing to my vain desire to be considered a courteous teacher."

The tendency toward a relaxation of discipline shows itself also in our laws. We congratulate ourselves on our humanity, because the long list of severe penalties, which formerly existed, has been replaced by a code much less severe; but have we not gone to the other extreme, encouraging a spirit of lawlessness and an unwillingness to submit to proper restraint? If one person shoots another with intent to kill, is the crime any less when the wound is slight than when it is fatal? Why is an attempt to wreck a railroad train, or to dynamite or set fire to a building which may involve the sacrifice of many lives, any less serious than to kill a man in a quarrel? Why should not the depraved individuals who commit acts like these be at once put out of the way for the good of society? Our nerveless laws and the misdirected efforts of inflamed philanthropy condone offenses all along the line. We coddle and sympathize and send flowers, where we should be stern and insist that the good of society is the first thing to be considered. Even in our daily dealings with individuals in matters not within the province of the law, we fail to realize that a prescription often needed, instead of coddling, kind words, sympathy, and self-sacrifice, is the prompt and energetic application of the toe of the boot to the lower end of the spinal column, repeated if necessary, until relief is obtained. We are victims to-day, it seems to me, of an exaggerated humanity, as well as of a decay of discipline. Large numbers of men and women with warm hearts and good intentions are exalting sentiment above sanity, sympathy above truth, and ease above discipline.

5.—Another of the tendencies very marked at the present time, is the spirit of innovation, which seems to pervade all things; whatever is, is wrong; the social custom must be changed in order to keep up with the time. This spirit is particularly observable among the young and the immature, especially if it has been preceded or accompanied by a lack of discipline in early training. It is an instance of what Ferrero terms "that youthful spirit of innovation which is in all ages the main source both of perversion and of progress." Many of our young men, "all aflame with an enthusiasm unquenched as yet by any continuous contact with affairs," set out to reform the world, not so much because it needs reform, as because they wish to justify their

existence. This spirit of innovation is at all times one to be expected and reckoned with. Every generation, as it comes along, will endeavor to change the condition in which it finds itself, and will designate that change with the name of progress, even though it be perversion. Oh! Progress! what crimes are committed in thy name! Even if a social condition were perfect and ideal, a new generation would endeavor to change it. Youth desires change; often because it knows no better. Age becomes conservative because it knows by experience the dangers of hasty or unwise change. The spirit of equality accelerates this tendency.

The United States is often spoken of as the country of the young man; the old are laid on the shelf just as soon as they become conservative, because wisdom, experience, and the authority of age are not respected. The adage, "old age for counsel, youth for action," is modified by the omission of the first half. Old age is not regarded at all, as soon as it desires stability and permanence rather than change. This tendency is certainly much more observable in this country than in Europe, where the wisdom which comes from age and experience is recognized and utilized. Gladstone and Voltaire did their best work after they were 60; Bismarck did his after he was 50. Gray hairs are common in the highest councils abroad. Here they are considered rather a symptom of decay.

This spirit of innovation which pervades all things seems to be largely a result of scientific progress; the rapid advances made in inventions of all kinds, the improvements in the arts and manufactures, have resulted in a complete revolution of the methods in many branches of industry within a comparatively few years. Men now living were born before there were any railroads, and almost all of us can remember the time when the applications of electricity were just beginning. An industrial plant but a few years old may be out of date and unable to compete with newer and more economical installations. In engineering matters we are accustomed to see things completely remodeled within a short time.

This familiarity with industrial change and the recognition of its benefits have naturally led to the idea that everything must be reconstructed, or it is old-fashioned and out of date, even though it be a thing in which there have been no new discoveries, warranting any change whatever: music, painting, dancing, education—everything, in

fact—must be completely reorganized. This willingness and desire for change afford the opportunity for unsuccessful mediocrity and for ambitious and unscrupulous failure to foist upon the public, which eagerly accepts it, forms of change which discerning men must characterize as gross, ridiculous, and demoralizing. The result, in many cases, is a deterioration pitiable to behold. Let any one compare, for instance, the modern painting, dancing, and music, with that in vogue fifty or one hundred years ago, and he will find abundant proof of this. In politics and government, too, the same tendency, the same recklessness, the same demoralization, are equally observable.

Now, of course, an existing condition is not necessarily the best because it is old; neither is a proposed change advisable because it is new. In many cases, however, suggested changes, ignorantly proposed by those unacquainted with history, have been tried and have been found wanting centuries ago. History repeats itself. Nations, like individuals, will only learn by costly experience. Away with the old, make way for the new! We will not profit by the lessons of our forefathers, we must experiment and learn for ourselves. Is it not much better that we should try to cultivate the breadth of view, the tolerance of opinion, which will lead us neither to adhere to that which is old, from extreme conservatism, nor to indulge in rash experiments; but to study history, cultivate a sound and deliberate judgment, and endeavor to guide the inevitable spirit of innovation into proper channels; and to remember, as Lecky well says, that "an appetite for organic change is one of the worst diseases that can affect a nation?"

6.—Advancing civilization and increase of wealth inevitably lead to increased luxury and extravagance; and, with our advancing civilization, this tendency is not wanting. If the experience of the past is of any value, however, it is a serious symptom of degeneration. The sudden accession of great wealth, particularly to those who are incompetent to use it wisely, engenders luxury and ostentation, and a contempt for those less fortunate; thus stimulating, to a certain extent, an attempt of the latter, particularly if endowed with the ballot, to recoup themselves and gain what they conceive to be their rights at the expense of wealth, by various forms of unjust taxation and other well-known expedients. If it is our duty to preach the gospel of content and the inequality of man, it is equally our duty to preach the proper use of wealth, and the proper attitude of the wealthy toward

those less fortunate; and while believing that individual initiative should be stimulated to the utmost, and allowed to enjoy the fruits of its success, it may, nevertheless, fairly be questioned whether some of the rewards under present conditions are not too large, and whether men would not exert themselves as well for the prospect of a more reasonable recompense as for the prospect of unreasonable and excessive riches. But, again history repeats itself, and things move in cycles. Peace causes prosperity, prosperity gives rise to wealth, wealth gives rise to luxury and extravagance, luxury and extravagance are the source of contention, contention results in war or social revolution, these result in poverty, poverty results in peace, and so the cycle is completed.*

Ferrero, writing of the condition of Rome in the Second Century, B. C., wrote:

"The destruction of Carthage and Corinth had already worked serious havoc in Roman life, spreading luxury and ostentation among the upper classes, a distaste for work among the lower, and wastefulness and intemperance in all ranks of society."

Do we not see before us to-day in America the same phenomena? It is not the destruction of Carthage and Corinth which have worked and are working serious havoc in our life, but it is the conquest of the virgin resources of a great continent. Wealth, luxury, and ostentation are the result, and nothing is more plain than the fact that a distaste for work exists and is spreading broadly among the lower classes, and wastefulness and intemperance everywhere.

7.—The last tendency to which I will call attention is what seems to me the distinct tendency, under present conditions, and, in fact, under conditions which have existed in increasing degree as civilization has advanced, toward the deterioration of the race. The law of evo-

*The following verse is taken from a collection of old French proverbs:

"Paix engendre prospérité,
De prospérité vient richesse,
De richesse orgueil et volupté,
D'orgueil contention sans cesse,
Contention la guerre addresse
La guerre engendre pauvreté,
Pauvreté humilité,
D'humilité revient la paix,
Aussi retournent les humans."

"The following passage from Ramsay's 'Voyages de Cyrus' contains a similar view with respect to the succession of causes and effects in the affairs of nations:—'Pendant l'espace de trois cens ans la valeur des rois de Médie avoit augmenté leure conquêtes. Les conquêtes avoient engendre de luxe, et le luxe est toujours l'avant-coureur de la chûte des empires. Valeur, conquêtes, luxe, anarchie, voila le cercle fatal, et les différens periodes de la vie politique de presque tous les états.'"

lution in all species of animals and plants, the law which we ourselves recognize and follow in breeding and cultivating them, is for the survival of the fittest, the gradual elimination of the weak and unfit. This is distinctly the law of progress. In the case of man, however, there is added an element which does not exist with the lower animals or with plants, namely, the element of human reason. Under the domination of this element, has not the tendency steadily been, with the growth of altruistic ideas and the prevalence of philanthropic institutions, toward the survival of the unfit? Are we not encouraging in many ways the survival of the weak, the inefficient, the incompetent, the depraved? If all men are equal, he who is born with an inheritance which makes him depraved or incompetent is not responsible for that inheritance, and is as much deserving of preservation as the strongest and the best.

Galton, in his researches on heredity, comes to the conclusion that the average Greek of the Athenian Age was as much superior intellectually to the average European of to-day, as the latter is superior to the average African savage. At the present time, in all probability, the uncivilized races are governed by the law of evolution, the survival of the fittest; while, in the case of the so-called civilized races, the reverse seems true. The two tendencies must meet and cross at some point in the future, if they continue.

This problem is one of the most serious that confronts us to-day. I have no time, even had I the knowledge, to discuss it in detail; I will content myself with merely mentioning it. It may seem wrong to believe that all have not equal right to survive, yet it seems to me that the interests of society as a whole must lead us to the conclusion that serious evil, and perhaps ultimate ruin, await us unless some means can be devised for solving this problem and in some manner regulating the perpetuation of our species. If present-day humanitarianism leads to race degeneration, it cannot be the way of progress intended by an all-wise Creator, and we should try to see in what way we are misapplying it. The subject is manifestly beset with great difficulties. In the first place, we must define the "fit." Clearly, the term is not synonymous with the physically strong, or the wealthy; many of these are most unfit. Equally clear is it that the fit do not include those afflicted with serious hereditary disease, the dishonest, the vicious, the intemperate, the indolent, the good-for-nothing. "If any will not work,

neither let him eat," says the Apostle. Those that will not work should starve, in other words. He adds: "For we hear of some that walk among you disorderly, that work not at all, but are busybodies." May it not be that the true main-spring of progress and prosperity is an enlightened selfishness—a selfishness which leads the individual to seek as his paramount object his own good, subject only to one limitation, that no injury should be done to any one else—that not self-sacrifice, but only a scrupulous regard for the rights of others, is the condition of race progress?

And now you may very likely have asked, what has all this to do with engineering, or with the engineering profession; what is the appropriateness of a discussion of social conditions before a society of engineers? My answer is that the engineer is primarily a member of the social body; that its problems are his problems; and that he cannot avoid the responsibility of taking a share in their solution. The problem of the Twentieth Century, it seems to me, will be pre-eminently a social problem, and upon its solution will depend the happiness of mankind for years to come. Moreover, this social problem is largely, as I have shown, the outcome of the work of the engineer who, as the advance agent of civilization, has been the main factor in creating the conditions which give birth to this problem. It will be evident, therefore, that I differ from one of my predecessors who, in his annual address, intimated that social problems were out of the province of the engineer and that he should not meddle with them. On the other hand, I maintain that it is his duty to exert himself in their solution. Moreover, I consider this duty the more necessary because I believe the engineer to be well fitted—possibly better fitted than any one else—to solve these problems wisely. Let me briefly give you my reasons for this opinion.

The engineer is by training a scientific man. If his education is what I conceive it should be, and what it will be more and more in the future, it will consist of a training in mathematics, logic, science, and in the technical branches of his profession, together with a sufficient amount of study of the humanities—history, economics, language, literature—to give him a clear view of the relations of things. He will be pre-eminently by training a scientific man, but with a power of understanding and grasping social questions. Now, a scientific training has this great virtue, that its end and aim is constantly

the discovery of the truth. The truth for its own sake, independent of everything else, is the object of the scientific man and therefore of the engineer. In this respect, it is, in my opinion, the noblest as well as the most widely useful of the professions. The lawyer is not always concerned in arriving at the truth; he is too frequently tempted to make the worse appear the better reason. As one of that profession has expressed it, his object is not to get at the truth but to win cases. He is constantly under the temptation to pervert the truth, or to suppress it, if the exigencies of his case demand.

Furthermore, the engineer is exposed to a very direct personal responsibility which cannot but steady his character and increase his self-reliance and self-control. Though he does not always receive the credit of his successes, he cannot succeed in throwing the responsibility for his failures upon others. If a lawyer loses his case, as Mr. Choate once remarked, it is never his fault; he can always charge it to the prejudice of the judge, or the ignorance of the jury, or the untruthfulness of witnesses. He goes on and obtains new clients and new fame. If the doctor loses his patient, it is never his fault; he can always charge it to the disobedience or delay of the patient, or maintain that a cure was impossible. But if an engineer builds a structure which fails or a machine which will not work, the responsibility is his and he is held accountable.

Again, the engineer is a business man, for engineering is business and business is engineering; he deals with men, he has to do with financial affairs, he learns the characteristics of human nature, and his training, therefore, tends to teach him tact, moderation, conservatism, and the value of experience.

Of course, any sweeping eulogy or disparagement of any class of men cannot be truthfully made. Engineers will differ on questions of opinion, as other men will. There are dishonest, untruthful, unreliable engineers, as there are men of this type in every class. No doubt the ranks of our Society include men who will widely disagree on the questions which I have discussed; men who will hold extreme views in one direction or in the other; men who will be dogmatic, intemperate in language and in thought, inaccurate in reasoning, insincere in opinion. No class of men has a monopoly of virtue or of error. Neither are these excesses confined to men of weak minds or deficient training. As Lecky says:

"Strange veins of insanity and capacities for enthusiastic folly sometimes flaw the strongest brains, and the impetuous ebullitions of youth which impel some men into extravagances of vice, develop in other natures into not less wild extravagances of thought."

Nevertheless, on the whole, if there is anything in training and experience, may we not believe that engineers, as a class, have the training, temper, and experience which will best enable men to judge sanely and solve wisely the social problems which confront us?

What, then, may the engineer do to aid in the solution of the social problems of the Twentieth Century?

In the first place, he should consider it his duty not to retire into the technical recesses of his professional work and content himself with being the servant of other men, but should actively exert himself and take the initiative in the great problems of the day. He will find that many of these problems in their details are largely of an engineering character.

What, for instance, is involved in the problem of administering the affairs of a great city? Do they not involve, in the main, the preservation of the health, safety, property, and order of the community? and are they not, therefore, predominantly of an engineering character? Financial and legal questions of great importance, of course, are included, but why should the engineering problems be considered secondary to these? Engineers can employ financiers and lawyers, as well as be employed by the latter. I see no reason why the position of mayor of a city could not be filled at least as satisfactorily by an engineer with proper breadth of view, training, and experience, as by a man of any other profession. Yet I doubt if the mayor of any large city in the United States is or has been an engineer.

The same holds true of other administrative positions. The management of our great corporations involves problems largely of an engineering nature, and high administrative positions in these concerns are being more and more given to engineers. Even such a position as that of president of a republic is not one which the engineer should necessarily consider himself excluded from. Why is he not as well qualified to fill it as a soldier, a lawyer, or a college professor, provided his training and experience, judgment and breadth of view warrant? Two of our greatest presidents, Washington and Lincoln, though not engineers, did have some little practice in what was then

termed engineering, and one president of the French Republic has been a professional government engineer.

There is perhaps a tendency to-day toward better recognizing the qualifications of the engineer in the administration of municipalities and larger political units. The position of commissioner of public works in a city is perhaps the most important municipal position next to that of mayor. Formerly, this position was monopolized in most cities by men who were mere politicians. Nowadays, it is quite common to appoint engineers to such posts. In Boston, Philadelphia, Cincinnati, and very likely other cities, these positions are now held by trained and educated engineers, and this recognition I consider significant and deserved. Engineers, however, will not and should not be recognized in this way unless they are as individuals equal to the responsibilities involved, unless they show their interest in public questions, unless they inform themselves about them, unless they study the economic and social, as well as the engineering problems involved, and unless they assert themselves before the public. This I trust they will do more and more as years go by.

"The fault, dear Brutus, is not in our stars,
But in ourselves, that we are underlings."

A question which is much before us at the present day is that of conservation. Conservation, as I understand it, consists in a wise economy and proper use of the materials and resources that are afforded us by Nature. It is essentially the avoidance of waste; it does not mean that natural resources shall be withdrawn from use, but that they shall be used wisely and economically. It does not mean that the present generation shall *suffer* for the benefit of future generations, but that the present generation shall not *rob* future generations. Now, the technical problem of the avoidance of waste and the proper use of natural resources is a problem already solved. Applied scientists can tell just what to do and how to do it. They know how to utilize the power of falling water and how to transmit it; they know how to utilize coal most economically, they know how to utilize forest products without waste, how to prevent the erosion of the soil, how to conserve health. The unsolved problem of conservation—the problem which now concerns us most—is not an engineering problem, but a social one. It is to alter the public state of mind, to make people thrifty and economical instead of extravagant and wasteful. It is

thus an attitute of mind that we have to deal with. If the attitude of mind of the public toward this problem could be made the correct one, if the individual man could be made thrifty, frugal, careful, thoughtful of the future, the problem of conservation would be solved. Why should not applied scientists, who know how to solve the problem technically, lead in educating the people to perceive the necessity for its solution? Should not mere self-interest, not to speak of public spirit, prompt us to do this?

In education, too, where, as I view it, the tendencies I have referred to are plainly evident, we may as a profession exert a good influence. I have often expressed the opinion that an engineering education is the best possible education for any vocation, because it combines, or should combine, in the best proportions, the study of abstract relations with that of natural science and of the humanities. These, combined with training and experience in the profession itself, should make the engineer as well educated as any class of men. He should be sane and sensible, not likely to be carried away by fallacious economic theories; conservative, and yet safely progressive. Engineers, I think, should exert more influence than they do in education, by actively interesting themselves in it, serving on school committees, and insisting on the maintenance of rigorous discipline and the scientific method. Above all, they should aid in maintaining the standard of engineering education, and prevent it from succumbing to the prevailing tendency to relaxation of discipline. The best students do not really seek such relaxation; and schools which make students work hard and which insist on a high standard of accomplishment, will attract strong men. The tendency of the times shows this, and proves that one movement to-day is toward utilitarian and vocational training. Some of the tendencies I have referred to may be in large measure only surface currents, and a strong undercurrent may perhaps be generated in another direction if the proper means are used. This is indicated by the growth of the engineering departments of our State universities within the last thirty years, and by the surprising development of such a school as the Massachusetts Institute of Technology from its first small beginnings to its position of recognized leadership in less than one-third of a century. Let us make the engineering schools the place of refuge for those who are opposed to the relaxation of discipline which prevails so largely elsewhere.

Finally, let us inquire what this Society, representing the Engineering Profession, may do in the directions which have been suggested. We have been criticized in the past as being inactive, as taking no part as a Society in large public questions. I must frankly confess that the criticism has seemed to me in some measure deserved. Our Society is large, rich, influential, but we have been too content to sit down in dignified ease, taking little or no initiative, and allowing other and younger societies to outstrip us in actual work performed and in real influence exercised. We have had some committees which have done good work in unifying practice in relation to technical matters. In these respects, however, we have been far surpassed by two or three of the newer societies, whose committees, after long and conscientious labor, have exercised far greater influence that we have in the solution of even professional problems. It is true that this is largely due to the fact that those societies are more specialized than ours, that they are composed of men all engaged in one line of work, who could co-operate and lay down the results of experiment, experience, and opinion more easily than we could. This Society, however, is the American Society of Civil Engineers, and includes engineers in all branches and from all sections of the country. There is no reason why the engineers in the different branches should not associate themselves together in sections, or why we could not do more committee work along the various engineering lines than we have done in the past.

I believe, therefore, that we should favor the establishment of branches of the Society in various parts of the country, and of student branches in the several technical schools; and that we should urge our committees to activity, supply them with funds, and perhaps pay the mileage of members who attend meetings. The Society is rich, and it is not a philanthropic institution. If members give their time to committee work, I see no reason why they should be expected to give money, and the expense of attending many meetings may not seldom be considerable.

Moreover, we have been very loath to take any steps in formulating answers to questions which are largely matters of opinion; and we have done, thus far, little or nothing to influence public opinion in regard to questions of the day. I believe that we could and should go much farther in these directions; that we should step forward, take

the initiative, and become a moving force in the community. Let me make some more definite suggestions.

In the first place, I believe we should have a standing Committee on Public Relations, its province being to keep track of all public affairs involving engineering questions directly or indirectly, in all the States in the Union, and to take such action as may be deemed desirable to insure a wise treatment of such questions. This committee might perhaps consist of the President and Secretary of the Society *ex officio*, of a certain number of members of the Board of Direction to be appointed each year by the President, and of a certain number of members of the Society not members of the Board of Direction, to be nominated by the Board of Direction. These outside members might be in considerable number, perhaps one or more from each State, or group of States, so that the committee could keep in touch with public affairs affecting the profession in all parts of the Union.

Further, our Committee on Engineering Education should, in my opinion, be enlarged, supplied with funds, and encouraged to active and exhaustive work. If we could lay down the principles which we believe should govern in engineering education, the general curriculum which should be adopted, the proportion of the humanities which should be included, and similar matters, we might, I believe, do much good, and, while still leaving ample room for individual initiative and divergencies in practice in different institutions, render effective aid to those teachers who are striving to resist some prevailing tendencies.

In the next place, I believe it will be wise to have more committees of the Society, dealing with specific problems, than we have had in the past. You have before you at this meeting, I am glad to state, a report from the Board of Direction stating that, as provided in the Constitution, it has voted to appoint two Special Committees, to deal respectively with the Regulation of Streams and with Uniform Water Legislation. I believe that still other committees could be appointed to advantage. We have no reason to be afraid of having too many committees. We should, rather, be sure that we lose no opportunity to exercise legitimate influence on public opinion or to aid in the solution of large problems. We might well, I believe, have a standing committee on conservation, perhaps another on corporation legislation, another on the history of American engineering. Let us give our earnest attention to this matter, and let us not hesitate to

suggest any and every means that may occur to us to promote the welfare of the Society, of the profession, and of the public. The Board of Direction will be always ready, I am sure, to act promptly with any members of the Society who can show a proper field for legitimate action.

Reviewing then, the points which I have brought to your attention, I have endeavored to show that the problem of the times is pre-eminently a social problem. I have outlined some of the tendencies of the day, as they appear to me. I have given the reasons for my belief that it is the duty of engineers to aid in the solution of these problems, and that by training and experience they are well qualified to do so. If my tone has appeared to indicate a pessimistic attitude toward the tendencies of the day, I fear I must plead guilty in some degree to the imputation; whether wisely or not, as the years have gone by, I have grown more conservative, and I find myself having little sympathy with many of the tendencies which I seem to see about me. I believe that this age will be looked upon by future generations, not as an age of progress, except in some material things, but more distinctively as an age of deterioration, an age of fads, frivolities, fancies, and follies; of much reading and little thought; an age of impulse rather than an age of reason. I am not alone, however, in a feeling of dissatisfaction. M. P. Leroy-Beaulieu, writing in 1890, and referring to conditions in France, expressed himself as follows:

"Every age is characterized by its particular craze. The present craze is for education, unlimited and injudicious, and for philanthropy equally unlimited and injudicious, both absolutely superficial. By their aid we have succeeded in producing a mental condition and in creating certain social circumstances which are most unfavourable to the growth of the population."

However, after all, it is only an epoch in the history of man. It will pass away, and others will solve the problems that we leave to them. Progress is not always continuous; there are waves, reactions, times of apparent retrogression. May we only do our part, as members of one of the greatest professions, to help direct the tendencies of our day in such manner that our successors may look back upon us with approval and not with blame, and that we may aid to the utmost in forwarding the true progress of mankind.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1267

A MECHANISM FOR METERING AND RECORDING THE FLOW OF FLUIDS THROUGH VENTURI TUBES, ORIFICES, OR CONDUITS, BY INTEGRATING THE VELOCITY HEAD.

By J. W. Ledoux, M. Am. Soc. C. E.

During the last century, and even prior thereto, investigations, both theoretical and practical, and many experiments were made in order to solve the problem of the flow of water through pipes, orifices, short tubes, etc. Venturi, an Italian investigator, made very interesting experiments about 1790, and devised what has since been called Venturi's tube, which, due to its peculiar shape, possessed the property of discharging water with a very small amount of friction. He, as well as later investigators, deduced formulas for the velocity and velocity head which do not differ much from those at present in use.

In 1887 and 1888 Clemens Herschel, M. Am. Soc. C. E., first suggested the practical use of Venturi's tube as a water meter.* His suggestion was far-reaching, because any device that could be used to record or integrate the velocity head of the Venturi tube would also perform that function for any orifice or contraction in a pipe or conduit wherein the flow is a function of the velocity head. Since that time many efforts have been made, with varying degrees of success, to design a registering mechanism which would indicate reliably the

* *Transactions*, Am. Soc. C. E., Vol. XVII, p. 228, and Vol. XVIII, p. 133.

total discharge through such a tube or orifice. Mr. Herschel has made several inventions of this character.

One of the most attractive devices for this purpose consists of a small meter in a by-pass connecting the full and contracted sections of a Venturi tube. Several engineers, including the writer, have independently conceived this idea, but afterward abandoned it as impracticable. At high velocities such a contrivance will give approximate results, but when the velocity is low, the velocity head, or moving force—which varies as the square of the velocity—is too minute to operate a moving mechanism.

Some of the early devices for indicating and recording the velocity head were based on Bourdon tubes, and in some cases diaphragms balanced by springs were used. One of Mr. Herschel's earliest devices was an ingenious apparatus based on the former principle; but, eventually, all these were abandoned in favor of mercury, which was found to be much more constant, reliable, and satisfactory as a transmitting medium.

The fundamental principles of the subject will be considered briefly. The flow of water and other liquids as well as gases follows the eternal and unchangeable law of gravity. For steady flow this may be represented by the expression: "Velocity is proportional to the square root of the head producing velocity," or, algebraically,

$$V = C\sqrt{H},$$

in which V = the velocity, in any convenient unit,

H = the head producing such velocity,

and C = a constant depending on the unit of measurement and the character of space through which the fluid flows.

V may be expressed in feet per second, miles per hour, etc., and H, the effective head, may be expressed in feet, or in pounds per square inch, etc.

The space may be a pipe of any length, a submerged orifice, an open inclined channel, a Venturi tube, or a nozzle. It is seen that this is the form of the simplest equation of a parabola:

$$y^2 = cx.$$

For practical purposes, the ideal form of meter would be a device, connected by small tubes with adjacent parts of a pipe through which fluid is flowing, which would measure automatically the square root

of the head or pressure producing the velocity. Such a device would offer no resistance or obstruction to the flow through the pipe. It has been impossible, heretofore, to achieve this ideal, because the forces due to velocity head are so minute as to be difficult, if not impossible, to measure or integrate when the velocity is low.

Theoretically, the Venturi tube is a contraction in the pipe, formed so as to cause small frictional resistance to flow; and it is remarkable how slight this contraction may be in order to produce the same difference in velocity head as that caused by Pitot tubes without contraction in the pipe. In fact, the diameter of the contracted portion need not differ from that of the full section by more than 17.7% in order to produce the full effect of two Pitot tubes, one pointing up and one pointing down stream, with their openings in the center of the pipe, or at the points of maximum velocity. In practice, however, the Venturi tube is composed of a short pipe continuously connected with the main pipe by a conical section pointing up stream and a longer conical section pointing down stream, the convergence of these two sections being dependent on experimental determinations, the object being to obtain the highest flow with the least frictional or unrecoverable loss of head through the entire Venturi tube. These angles of convergence and divergence have been determined by Venturi and, in more modern times, by Weisbach, Eytelwein, Francis, Smith, Herschel, and possibly others, but, at any rate, it is pretty well settled that they should be somewhere between fairly wide limits, beyond which the results are certainly not as good. However, they may be relatively the same for all Venturi tubes, whether the contraction is nearly as large as the pipe or very much smaller, the difference being that in the former case the entire Venturi tube would be short and in the latter case long. Of course, the purpose of a Venturi tube, the contracted section of which is small as compared to the full bore of the pipe, is to exaggerate the velocity head indications so that they can be observed and measured, especially for low flows. To illustrate, consider the simple formula of a standard Venturi tube, having a coefficient of velocity = 0.977:

$$V = 7.84 \left(\frac{H}{\left(\frac{D}{d}\right)^4 - 1} \right)^{\frac{1}{2}}$$

in which the only two variables are V and H.

V = the velocity through the full section, in feet per second;
H = the difference in piezometric heads between the up-stream section and the contracted section, in feet;
D = the diameter (or radius) of the full section;
d = the diameter (or radius) of the contracted section.

This formula may be written:

$$H = 0.0163\ V^2\left(\left(\frac{D}{d}\right)^4 - 1\right).$$

Now, assume a velocity in the main as low as 0.25 ft. per sec. Then the formula may be written

$$\left(\frac{D}{d}\right)^4 = 1 + 982\,H;$$

and assume 0.01 ft. to be as low as it is possible to indicate H; then

$$\frac{D}{d} = \sqrt[4]{10.82} = 1.86,$$

which means that when the velocity through the main is as low as 0.25 ft. per sec., if it is desired to get an indication of the velocity head as great as 0.01 ft., the contracted section of the Venturi tube must be less than 54% as large in diameter as the main. However, with a tube having ratios of diameters equal to 3 to 1,

$$H = 0.0815 \text{ ft. for } V = 0.25 \text{ ft. per sec.}$$

Even this result cannot be obtained with the best apparatus formerly in use, unless one of the essential features of the Venturi meter is seriously sacrificed, namely, the measurement of high velocities.

The best types of Venturi meter registers have formerly depended for their registration on some form of cam having a parabolic curve, fashioned around a drum or revolving about a center. Near zero, or the origin, the curve changes in direction rapidly, and at zero crosses one of the axes of reference at right angles and becomes parallel to the other. It is probably this feature, more than any other, which has limited the minimum velocity measurable by a Venturi meter; and though the measurement of low velocities is attainable by a high ratio of tube diameters, there is thus encountered the other difficulty of excessive frictional loss through the Venturi tube for comparatively high velocities. If it is even intended to measure velocities as high

as 10 ft. per sec., which is occasionally desirable, the frictional head through a Venturi tube having a ratio of 3 to 1 in diameters, or 9 to 1 in areas, would be 17 ft., obtainable from the formula:

$$H_f = 0.0021\ V^2_2,$$

in which H_f = the frictional head, in feet,

and V_2 = the velocity through throat, in feet per second.

Besides that, in any form of register depending on mercury as the medium of indicating the velocity head, in order to cover ranges of velocity from 10 ft. per sec. down to zero, the apparatus would have to be more than 11 ft. high. In a tube having a ratio of 2 to 1 in diameters, or 4 to 1 in areas, with a velocity of 10 ft. per sec. through the main, the frictional head would be only 3.36 ft. This is still a material loss, but a velocity of 10 ft. per sec. is unusually high.

It must be remembered that, as far as metering a fluid is concerned, any form of orifice or contraction in a pipe would be just as good as a Venturi tube. It is also highly probable that the coefficient of flow would remain as nearly constant through just as wide a range of velocities; but the great advantage of a Venturi tube is in its small frictional resistance to flowing fluid, and in this respect it is the best shape yet devised.

The writer will now describe a form of meter register particularly adaptable to the registration of flows through any form of orifice, such as by the use of Pitot tubes and Venturi tubes of small ratios and consequently minimum frictional heads. The fundamental principles were discovered by the writer in 1904 and were finally developed into the best practical working form in 1909. The basic idea is a float, shaped so that, when acted on by the head due to the velocity of flowing liquid, its movement will be directly proportional to that velocity.

The device is novel in form, and covers a principle in hydraulics, which, although rational and perfectly obvious on analysis, the writer believes has not heretofore been recognized. This may be stated as follows: If a hollow body of any shape, with open bottom, floats freely in a liquid having the same specific gravity and touches another liquid of greater specific gravity, and if the relative pressures without and within this hollow body are changed so as to make the internal pressure less than the external pressure, then the body will sink into the heavy liquid, and the amount of displacement will be exactly

equal to the quantity of the heavier liquid which will pass up into the interior of the body and above the level of the heavier liquid outside of the body.

Let the hollow body, or float, be represented by *n n a a*, Fig. 1; assume it to be a figure of revolution, and call the total volume of heavy liquid, *a a m m*, within the float *M*.

The principle may be understood by a consideration of the laws of equilibrium—the algebraic sum of the vertical forces must be equal to zero; the sum of the horizontal forces, and the sum of the moments must all be equal to zero. We need consider only the first condition as affecting the problem. At the plane, *b c c b*, the heavier liquid is subject to the same unit pressure within and without the hollow body, but the resultant of external forces acting downward on the hollow body is measured by the weight of the heavier liquid, *m m c c*, and the total upward force acting on the body is the displacement of the annular portion of the body, *bca-bca*. For equilibrium, the algebraic sum of these two forces must be equal to zero, and, as one is positive and the other negative, they must be equal in amount; but, if that is the case, the entire quantity of liquid equivalent to the displacement of the hollow body, and no more, must be found in the interior of the hollow body, and hence the outer level of the surface of the heavy liquid does not change with variations of pressure outside and

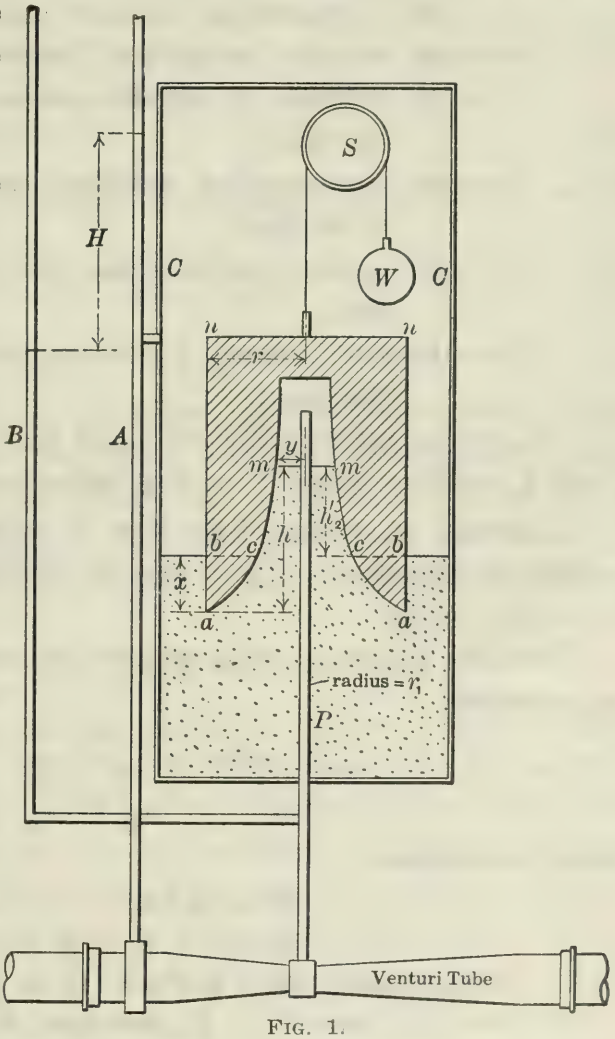


FIG. 1.

inside the float.* If there is a difference of pressure, H, then the float will sink into the heavier liquid an amount represented by the annular volume, acb, which will be called V; and the net volume of liquid $m\ m\ c\ c$, will also be equal to V, and h_2 will be proportional to H. As before stated, the level bb, will always be the same, no matter what the values of H, h_2, and V may be.

Let x = the vertical movement of the float from a given datum;
H = the Venturi head;
h_2 = the corresponding mercury head;
v = the velocity through the Venturi tube;
y = the ordinate or variable radius of the inner portion of the float;
h = the corresponding abscissas, measured from the bottom of the float;
r = the constant radius of the outer cylindrical portion of the float;
M = the total volume, $a\ a\ m\ m$, of the heavy liquid within the float;
V_0 = the volume of heavy liquid above cc;
and V_1 = the volume of the float below $bc\ cb$.

Assume the radius of the pipe, P, to be small and negligible.

Take, by hypothesis, $v \propto x$, also for unity coefficient, $x^2 = h_2$, for $v \propto \sqrt{H} \propto \sqrt{h_2}$.

Therefore, the cross-section of the float must conform to the following equation:

$$y^2 = \frac{r^2}{2\sqrt{h + \frac{1}{4}}}$$

derived as follows:

$$H = 12.6\ h_2$$

$$h = x + h_2 = x + x^2.$$

Geometrically, $M = \pi\ r^2\ x + V_0 - V_1$

but $V_0 = V_1$, therefore, $M = \pi\ r^2\ x$ ..... (6)

Differentiating, we have $\frac{dM}{dx} = \pi\ r^2$ ..... (7)

* If the float were not free to move vertically—that is, if it were restricted in any way, as with a variable lever arm, counterweight, or spring—then the movements of the liquids would not follow the law stated; but, as the difference of pressure became greater, the surface level of the heavy liquid would fall below a fixed level on the outside and rise above a fixed level on the inside.

Also, by the differential equation of cubature,

$$M = \pi \int_{h=0}^{h=x+x^2} y^2\, dh \qquad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Differentiating, we have $\frac{dM}{dx} = \pi\, y^2\, (1 + 2\, x)$ $\ldots\ldots\ldots\ldots\ldots\ldots (9)$

Combining Equations (7) and (9) and reducing, we have

$$y^2 = \frac{r^2}{1 + 2\, x} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$\text{but } h = x + x^2 \qquad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

$$\text{hence } x = -\frac{1}{2} \pm \sqrt{h + \frac{1}{4}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

and as $h = 0$ when $x = 0$, we must use the plus sign before the radical, therefore,

$$y^2 = \frac{r^2}{2\sqrt{h + \frac{1}{4}}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

This is an equation of the fifth degree, as between the two variables, h and y. No attempt has been made to discuss this equation exhaustively or plot all the loci, for, though this would be interesting from a purely mathematical standpoint, it is evident that only that locus will satisfy the physical conditions wherein both h and y have positive, real values.

For $h = 0$, $y = r$; and for $h = \infty$, $y = 0$,

thus it is seen that there are no values of h between 0 and ∞ which give infinite or imaginary values of y. As a consequence, the integrating apparatus operated by the sheave, S, is theoretically accurate from zero up to any practical limit that may be desired.

The shape of the float is such that its vertical movement is proportional to the square root of h_2 or H (the mercury and water Venturi heads, respectively); and, as the velocity through the Venturi tube is also proportional to the square root of H, the vertical movement is directly proportional to the velocity or quantity of water passing through the Venturi tube. The same principle will hold true if the Venturi tube is replaced by any form of orifice, or if the pipes, A and B, be connected with Pitot tubes in a main; for, in any of these cases, the H values represent velocity heads, the same as they do for Venturi

tubes, and the velocity heads are always proportional to the square of the velocity producing the head.

It is perfectly practical to have x^2 less or greater than h_2, for, if we assume $cx^2 = h_2$, Equation (13) becomes:

$$y^2 = \frac{r^2}{2\sqrt{c}\sqrt{\frac{c}{4} + h}} \qquad (14)$$

which is the more general form, in which we can give to c any value, consistent with practical limitations. For general use, however, Equation (13) is satisfactory, particularly as there is no value of c which increases or diminishes the accuracy of the apparatus.

In these calculations, the radius of the pipe, P, within the float, is assumed to be so small that it does not affect the results. Even if no correction is made for this volume, the error at low velocities would be negligible, and, at maximum velocities, it would be less than half of 1%, provided the pipe is made as small as possible. Its effect is to raise slightly the level of the heavy liquid, both inside and outside the float, by the same amount and aggregating the volume of displacement of the pipe, P; and, where it is desired to make this inside pipe, P, of an appreciably large outside diameter, allowance must be made for its disturbing influence. The final rationally derived formulas for that purpose are as follows:

$$x^2 = h_2$$

$$h = x + x^2 + \frac{r_1^2\, x^2}{y^2 + R^2 - r_1^2 - r^2}$$

$$x = \frac{(r^2 - y^2)\,(y^2 + R^2 - r_1^2 - r^2)}{2\,y^2\,(y^2 + R^2 - r_1^2 - r^2) - 2\,r_1^2\,(r^2 - y^2)}$$

in which the only variables are x and y,

h_2 = the mercury rise as before;

x = the vertical movement of the float;

and y = the variable radius of the interior.

The constants are:

R = radius of cylinder containing float;

r = radius of outer surface of float;

and r_1 = radius of pipe P.

To use these formulas practically, it is necessary to tabulate x and y, h_2, and finally h; h representing the abscissas, as before, and y the

ordinates. The necessity of this complication can be avoided by enlarging the outer cylinder at the location of the surface of the mercury.

Although the writer has devised many other forms and modifications, the form of float illustrated in Fig. 1, and covered by Equations (13) and (14), is believed to be by far the most practical, because it provides for maximum increments of volume and consequent accuracy at minimum velocities; and, with a float of a given radius, the minimum clearances are possible with the smallest cylindrical casing; also, there are several other practical advantages of importance. When this paper was first prepared it contained a discussion of every practical modification of form, with the derivation of the equation of each float. These, however, would take up too much space and be of mathematical rather than of mechanical interest.

At first thought, it might seem that a more practical shape would consist in a cylindrical interior and a curved exterior, because that would be formed more readily and accurately. Its cross-section would be a curve of the second degree, and the volume would be a paraboloid of revolution, convex outwardly. However, it cannot be made with greater accuracy, or probably with greater facility, and, for an apparatus of the same size, the form shown in Fig. 1 is almost seven times as accurate at minimum velocity.

The essential requirements of a water meter are:

1st. Accuracy within the range of the apparatus.
2d. Range, or ability to measure accurately flows in which the percentage of the maximum is a minimum.
3d. Minimum frictional resistance to flow.
4th. Reliability, or ability to perform with the greatest continuity, the minimum of attention, and the least danger of accident or damage.
5th. Long life and small repairs.
6th. Low cost.
7th. Simplicity.
8th. Accessibility—easy to examine and adjust.
9th. Minimum dimensions and weight, so as to occupy the least space.
10th. Good appearance.

The apparatus having the principles shown in Fig. 1, and described more in detail herein approximately fulfills these ten requirements.

One of the first difficulties to overcome was to prevent air from accumulating at the upper interior of the float in Fig. 1. Such an accumulation, of course, would lighten the float and therefore vitiate the results. Two check-valves opening upward are placed in the cap of the float. These are opened automatically when the velocity head reaches the maximum, and they can be opened at will, by a hand device, from the outside of the bottom of the pressure chamber, without otherwise dismantling the meter. This same device also automatically prevents the loss of mercury when the flow reaches an amount beyond the maximum capacity of the meter register, as it equalizes the pressure within and without the float.

An additional device for this purpose has been designed to prevent loss of mercury due to a sudden break of either of the pipes leading from the meter register to the Venturi tube, but these are safety devices of not unusual novelty, and, therefore, a detailed description of them is unnecessary.

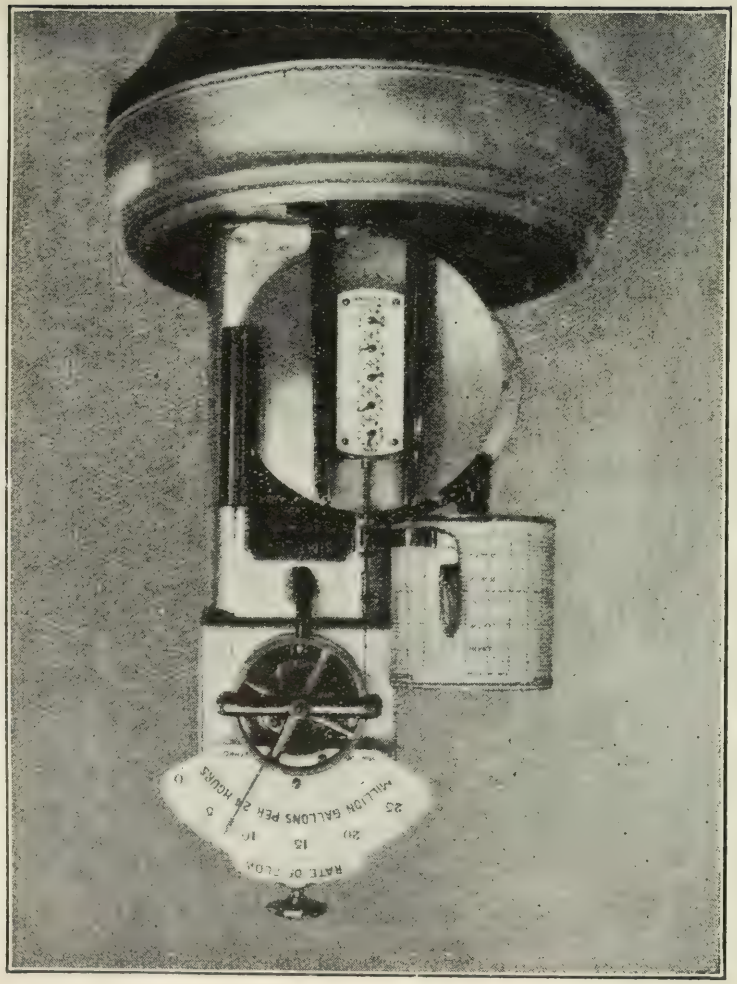
Figs. 2 and 3 show front and side views of this meter register.

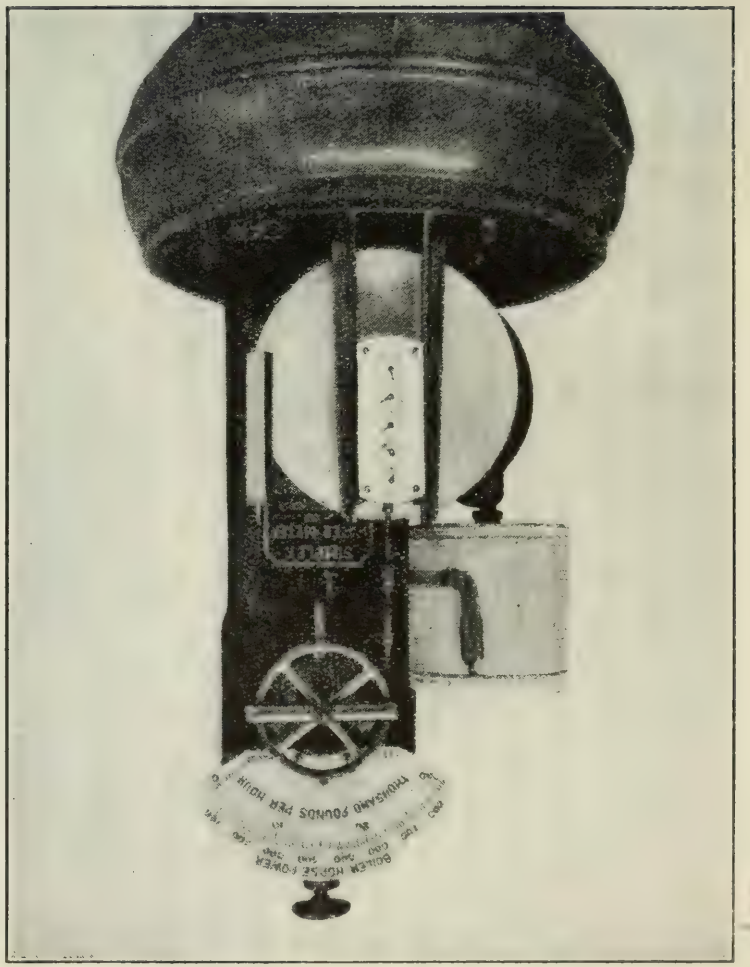
The float is made of hard rubber to conform to a metal template; but, to make sure that it is correct and will perform according to the theory on which it is designed, each float is tested in a regular working machine by applying, under pressure, piezometric heads corresponding to the velocity heads of the Venturi tube or Pitot tube with which it is to be used, and noting upon a standard rate-of-flow dial the corresponding velocities, which are compared with the theoretical velocities.

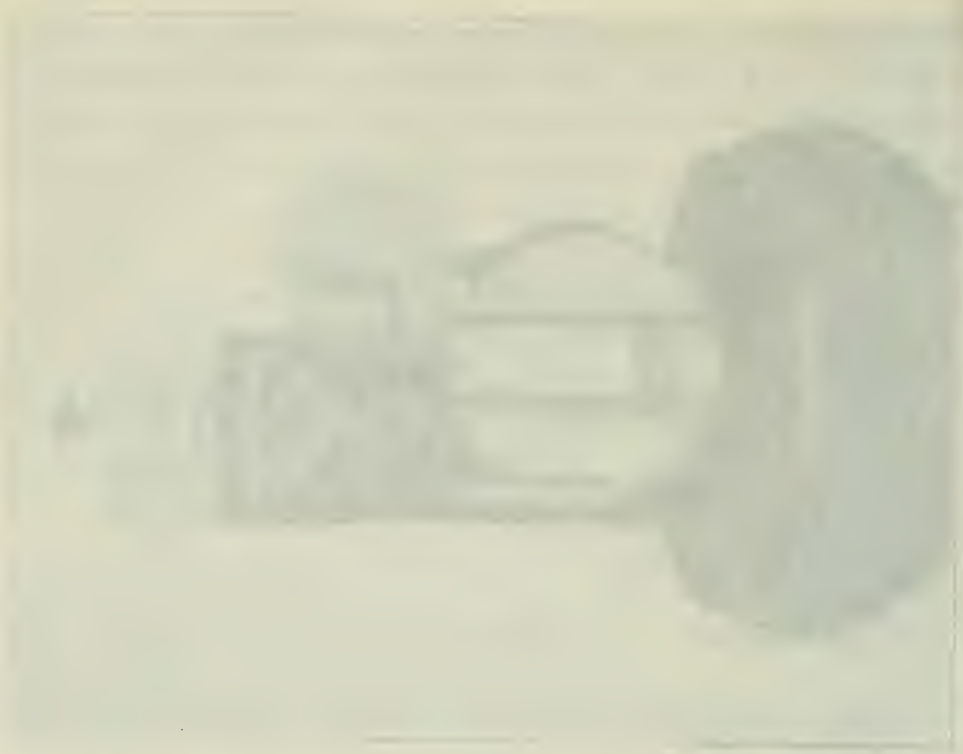
For heads, H, greater than $\frac{1}{8}$ ft. the coefficient of velocity through a standard Venturi tube is so nearly constant as to secure results within 2% of the actual. However, with the apparatus above described, Venturi heads, H, of less than 0.01 ft. are readily integrated. The velocity coefficient through the Venturi tube, however, is materially different, amounting to as much as 7% at low velocities, and the instrument is modified accordingly so as to produce accurate results down to less than 0.01 ft.

The data for this modification, as well as the method of making the correction, are intended to be the subject of a later paper.

Leading up to the final development of the apparatus illustrated in Fig. 1, it will be interesting to consider two of the earlier forms. Fig. 4 shows an extremely simple apparatus, which, at first sight, would appear to be superior to the device shown in Fig. 1.


RATE OF FLOW
MILLION GALLONS PER 24 HOURS


THOUSAND POUNDS PER HOUR
BOILER HORSE POWER



P is a piston, F is a shaped float, S is a sheave, and W is a displacement device, all within a casing. The upper portion of this casing is connected with a pipe from the full section of the Venturi tube, and the portion below the piston, P, is connected with a pipe leading to the throat of the Venturi tube. The differential pressure, or Venturi head, H, acts on the piston and forces the float, F, down into the mercury. As before, the problem is to shape the float so that its vertical movement from a fixed line of reference will be proportional to the square root of H. The displacement device, W, is such that when there is no flow in the Venturi tube and the apparatus is in equilibrium, the bottom of the float, a, will be tangent to the upper surface of the mercury. The level of the mercury, $m\ m$, will change, of course, as the float descends. The equation of the float, though rather more difficult to deduce, is as follows:

$$y^2 = e - \frac{f}{\sqrt{h+g}}$$

e, f, and g being constants depending on the size of the piston, P, the assumed movement of the float, F, with the given Venturi head, H, and the relative weights of mercury and water. The only radical difference in form between this formula and the one corresponding to Fig. 1 is in the constant, e, before the fraction. The curve, however, is still one of the fifth degree, and in this case when h becomes zero, y becomes zero, for the actual formula is:

$$y^2 = \frac{a_1}{\pi} - \frac{1}{2 \times 0.454\ \pi} \sqrt{\frac{\frac{(0.454\ a_1)^3}{0.434\ c\ a}}{h + \frac{0.454\ a_1}{4 \times 0.434\ c\ a}}}$$

in which a_1 = the area, in square inches, of the cylinder containing the mercury;

a = the area of the piston, P;

h = the depth of submergence of the float in mercury, in inches;

c = the ratio of the square root of the Venturi head to the velocity;

0.434 = the ratio of the pressure, in pounds per square inch, to the Venturi head, in feet;

0.454 = the weight per cubic inch of mercury in water.

The objection to this apparatus is the friction of the piston, *P*. This was recognized immediately, and, to overcome it, a diaphragm of an accordion or some very flexible type was substituted for the piston; this, however, introduced difficulties caused by the elasticity of the diaphragm and the necessity of modifying the shape of the float to provide for these variations, as the diaphragm could never be depended on to remain in a permanent condition.

Another form is shown in Fig. 5. This is on an entirely different principle, but the treatment of the problem is analogous to that of Fig. 1.

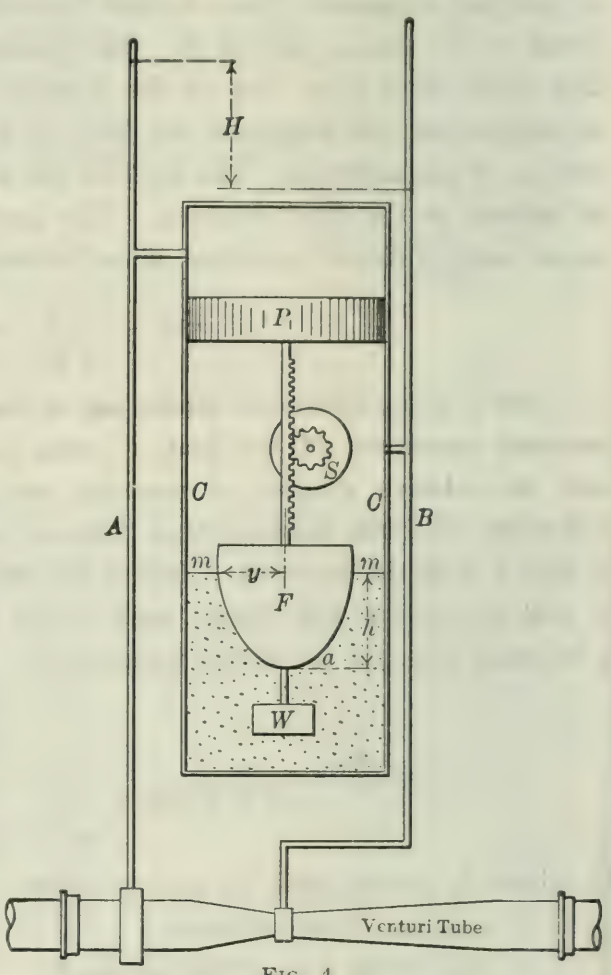


FIG. 4.

C represents a casing within which is contained a fixed displacing device, *F*, and a movable cylindrical vessel, *P*, containing, say, mercury buoyed up by the displacing volume, *W*. The pipe leading from the full section of the Venturi tube connects as shown above the vessel, and the pipe leading from the throat of the Venturi tube connects within the displacing device, *F*. The differential pressure, or Venturi head, *H*, forces mercury from the outside of the displacing device, *F*, to the inside; but, for equilibrium, the depth of the mercury in the vessel, *P*, must always be the same; that is, *d* must always be a constant. With these data it is found that the equation representing the inner curve of the displacing device is exactly the same as that of the float in Fig. 1, namely:

$$y^2 = \frac{r^2}{2\sqrt{h + \frac{1}{4}}}.$$

This device, in some respects, is superior to any of the others considered, and, in fact, it was the one of which a complete working model was first made and tested, and the results were good. The only objection is the large quantity of mercury required for the displacing device, *W*. It is obvious that the vertical movement of the cylindrical vessel, *P*, may be guided by rollers, as shown, or in some other practical manner. The cross-section of the rod, *l*, need not be sufficiently large to make its variable displacement affect the results appreciably.

However, the form in Fig. 1 is that which has been adopted as the most practical; and of all the twelve other devices, each of which has been developed by the writer to a working design, that shown in Fig. 5, next to that in Fig. 1, is the most meritorious. In all cases the registering mechanism outside of the casing may be exactly the same as shown. It comprises a rate-of-flow dial, a chart device to record the rate of flow, and a total-flow dial. The first requires only a graduated dial and a hand actuated by the shaft of the sheave, *S*. The second requires a pen reciprocating along the surface of a revolving figure having straight-line elements, such as a cylinder. The third requires a dial having hands which always revolve at a speed proportional to the angular position of the sheave, *S*. This is readily accomplished by hanging from the sheave, *S*, the dial which is connected with and operated by a traction wheel, the latter

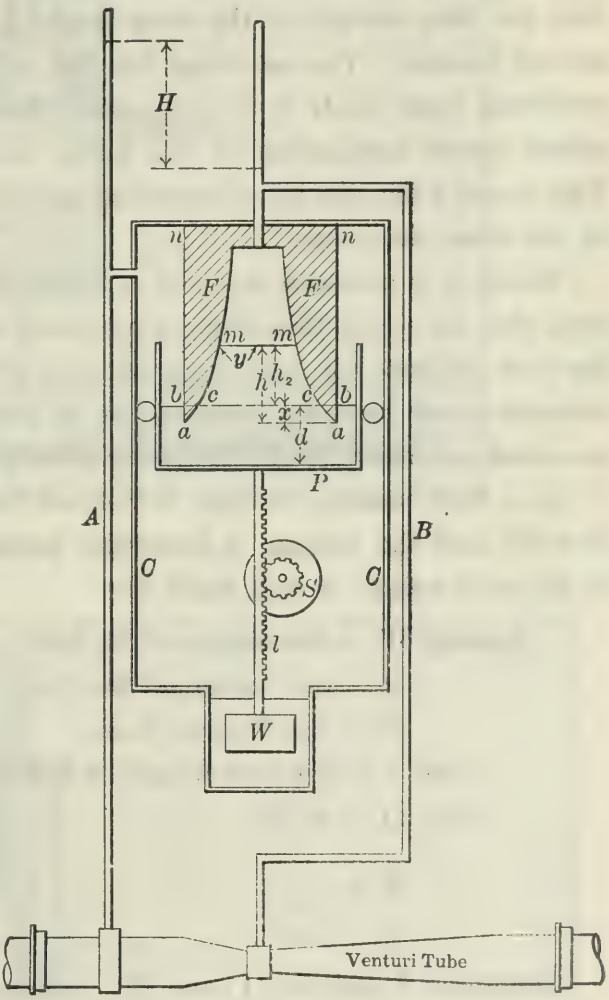


FIG. 5.

being rotated by a constantly revolving disk. When the sheave, S, sets at zero, the traction wheel stands at the center of the disk, and therefore has no rotating motion. As the flow increases from zero and the sheave, S, revolves, the traction wheel moves away from the center of the disk and therefore revolves at a speed proportional to its distance from the center of the disk and to the rate of flow through the Venturi tube.

There are other equally simple means of actuating the total-flow dial, but they are all on the same principle, and have been in use for several decades. The revolving flat disk is probably the oldest, but the revolving cone rivals it in antiquity. The Nicholson ship's log is a rather recent application of the latter device. It will be seen from Figs. 2 and 3 that the chart-recording cylinder and the disk are operated by the same clockwork.

Where it is intended to meter a liquid of a specific gravity different from that for which this meter is adjusted, as for instance, if the meter has been adjusted for measuring water at 50° Fahr., and it is desired to measure water having a temperature of 200° Fahr., the results can be corrected according to the following principle:

In a fluid passing through a Venturi tube or orifice, the weight is directly, and the volume is inversely, proportional to the square root of the unit weight of that fluid, for:

Letting W = the weight of the fluid,
Q = the volume of the fluid,
H = the Venturi head,
and w = the unit weight of fluid;

then $$Q \propto \sqrt{H} \quad \ldots\ldots (1)$$

$$H \propto \frac{1}{w} \quad \ldots\ldots (2)$$

$$W \propto w\,Q \quad \ldots\ldots (3)$$

Combining Equations (1) and (2), we obtain

$$Q \propto \sqrt{\frac{1}{w}} \quad \ldots\ldots (4)$$

Combining Equations (1), (2), and (3), we obtain

$$W \propto \sqrt{w} \quad \ldots\ldots (5)$$

Equations (4) and (5) being the stated relations.

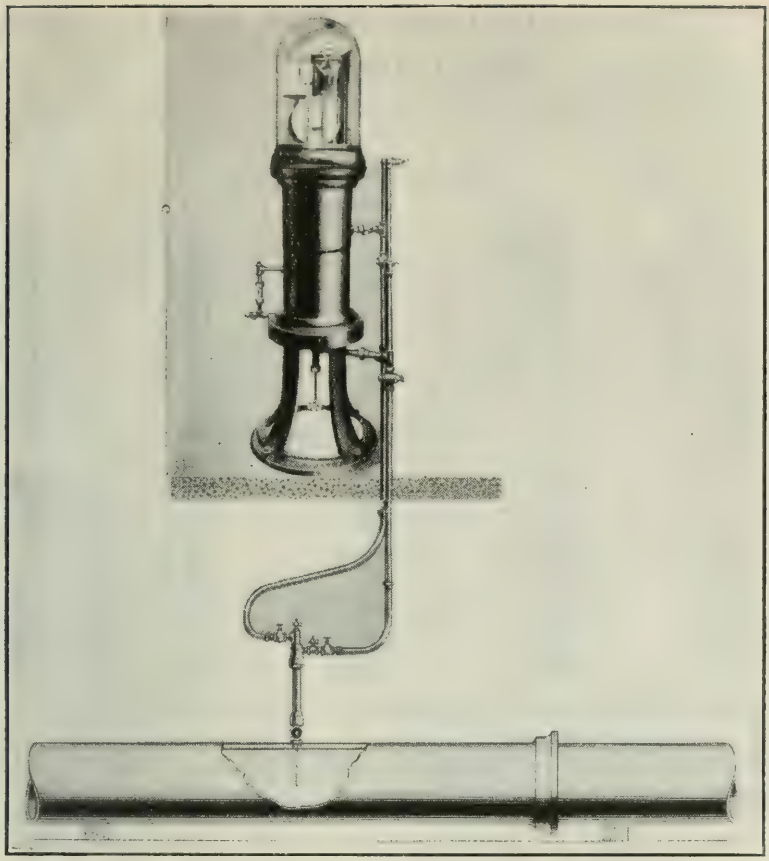


FIG. 6.—STATIONARY METER WITH PITOT TUBE.

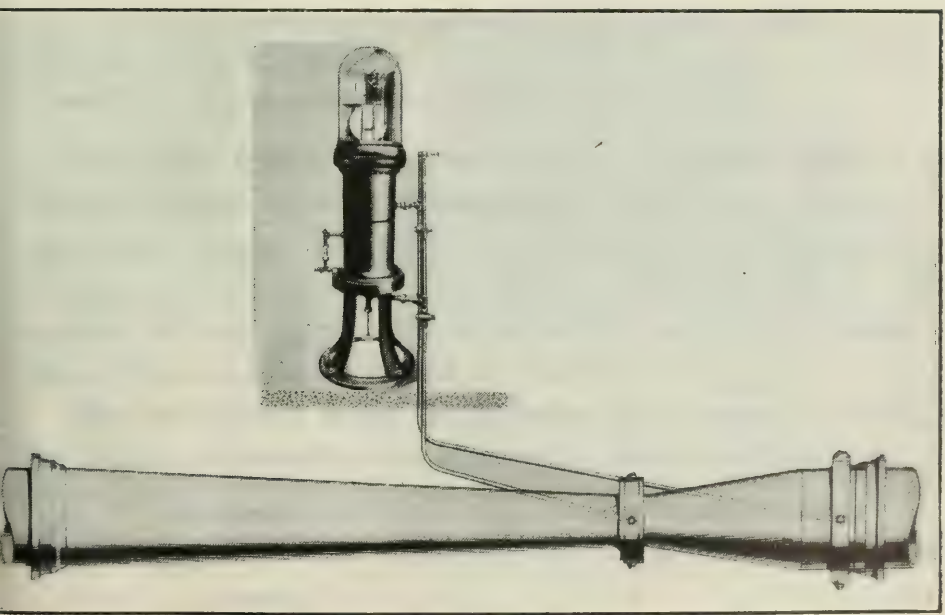


FIG. 7.—STATIONARY METER WITH VENTURI TUBE.

TABLE 1.—Test of a Meter Register Based on Venturi Heads. Float No. 46.

C	V	V_1	D	P	Date
0.010	0.195	0.194	+ 0.001	+ 0.52	
0.013	0.226	0.225	+ 0.001	+ 0.45	
0.023	0.300	0.298	+ 0.002	+ 0.67	
0.036	0.381	0.379	+ 0.002	+ 0.53	
0.054	0.470	0.467	+ 0.003	+ 0.64	
0.078	0.571	0.562	+ 0.009	+ 1.60	
0.112	0.677	0.675	+ 0.002	+ 0.30	
0.154	0.800	0.797	+ 0.003	+ 0.38	
0.208	0.931	0.930	+ 0.001	+ 0.11	
0.279	1.079	1.080	− 0.001	− 0.09	
0.368	1.240	1.240	0.000	0.00	August 29th, 1912.
0.487	1.429	1.428	0.001	0.00	
0.638	1.635	1.635	0.000	0.00	
0.833	1.872	1.874	− 0.002	− 0.11	
1.092	2.150	2.133	+ 0.007	+ 0.33	
1.426	2.460	2.460	0.000	0.00	
1.873	2.820	2.825	− 0.005	− 0.18	
2.455	3.250	3.235	+ 0.015	+ 0.47	
3.250	3.750	3.720	+ 0.030	+ 0.81	
4.320	4.321	4.280	+ 0.041	+ 0.96	
4.830	4.570	4.540	+ 0.030	+ 0.66	
5.800	5.010	4.970	+ 0.040	+ 0.81	August 28th, 1912.
7.270	5.638	5.600	+ 0.038	+ 0.68	

Tables 1 and 2 contain data relating to typical tests of meter registers constructed on the principle described herein, in which

C = the Venturi head as measured, in feet;

V = the theoretical velocity through the full section of the Venturi tube, in feet per second;

V_1 = the velocity as indicated by the meter register;

D = the differences;

and P = the percentage of error.

This meter register is in use in several important cases for recording and integrating the flow measured by Pitot tubes, and has also recently been adapted in portable form utilizing Pitot tubes for city main leakage survey purposes. This form is shown in Fig. 8. Quite a number of these instruments are already in service. A typical test of one of these recorders is shown in Table 3.

The first column shows the direction of movement, U being where the column is increasing and D where the column is decreasing.

C represents the Pitot tube velocity heads, in feet;

V represents the theoretical velocity as calculated for the Pitot heads;

V_1 represents the velocity as measured by the Pitot recorder;

D represents the differences;

and P represents the percentage of error.

TABLE 2.—Test of Meter Register.

Float No. 174.

Up or down	C	V	V_1	D	P	Date
U	0.0026	0.0890	0.0960	—0.007	—7.39	
D	0.0026	0.100	0.0960	+0.004	+4.17	
U	0.0043	0.115	0.1230	—0.008	—6.50	
U	0.0064	0.140	0.1520	—0.012	—7.89	
D	0.0064	0.145	0.1520	—0.007	—4.61	
U	0.009	0.186	0.1900	—0.004	—2.11	
U	0.013	0.218	0.220	—0.002	—0.91	
D	0.013	0.219	0.220	—0.001	—0.46	
U	0.036	0.372	0.372	0.000	0.00	
D	0.036	0.372	0.372	0.000	0.00	
U	0.054	0.458	0.457	+0.001	+0.22	
D	0.054	0.457	0.457	0.000	0.00	
U	0.111	0.658	0.660	—0.002	—0.33	
D	0.111	0.658	0.660	—0.002	—0.30	
U	0.154	0.775	0.779	—0.004	—0.51	
D	0.154	0.775	0.779	—0.004	—0.51	
U	0.208	0.910	0.910	0.000	0.00	
D	0.208	0.910	0.910	0.000	0.00	
U	0.279	1.054	1.056	—0.002	—0.19	
U	0.368	1.210	1.216	—0.006	—0.49	
D	0.368	1.210	1.216	—0.006	—0.49	
U	0.487	1.390	1.395	—0.005	—0.36	
D	0.487	1.390	1.395	—0.005	—0.36	
U	0.638	1.595	1.600	—0.005	—0.31	
D	0.638	1.595	1.600	—0.005	—0.31	
U	0.833	1.820	1.833	—0.013	—0.71	
D	0.833	1.823	1.833	—0.010	—0.55	
U	1.092	2.090	2.105	—0.015	—0.71	December 30th, 1912
D	1.092	2.087	2.105	—0.018	—0.85	
U	1.426	2.395	2.403	—0.008	—0.33	
D	1.426	2.397	2.403	—0.006	—0.25	
U	1.873	2.753	2.760	—0.007	—0.25	
D	1.873	2.752	2.760	—0.008	—0.29	
U	2.455	3.158	3.160	—0.002	—0.06	
D	2.455	3.155	3.160	—0.005	—0.16	
U	3.250	3.640	3.635	+0.005	+0.14	
D	3.250	3.637	3.635	+0.002	+0.06	
U	4.320	4.210	4.183	+0.027	+0.64	
D	4.320	4.210	4.183	+0.027	+0.64	
U	5.800	4.887	4.860	+0.027	+0.56	
D	5.800	4.895	4.860	+0.035	+0.72	
U	7.890	5.710	5.680	+0.030	+0.53	
D	7.890	5.705	5.680	+0.025	+0.44	
U	9.390	6.210	6.200	+0.010	+0.16	

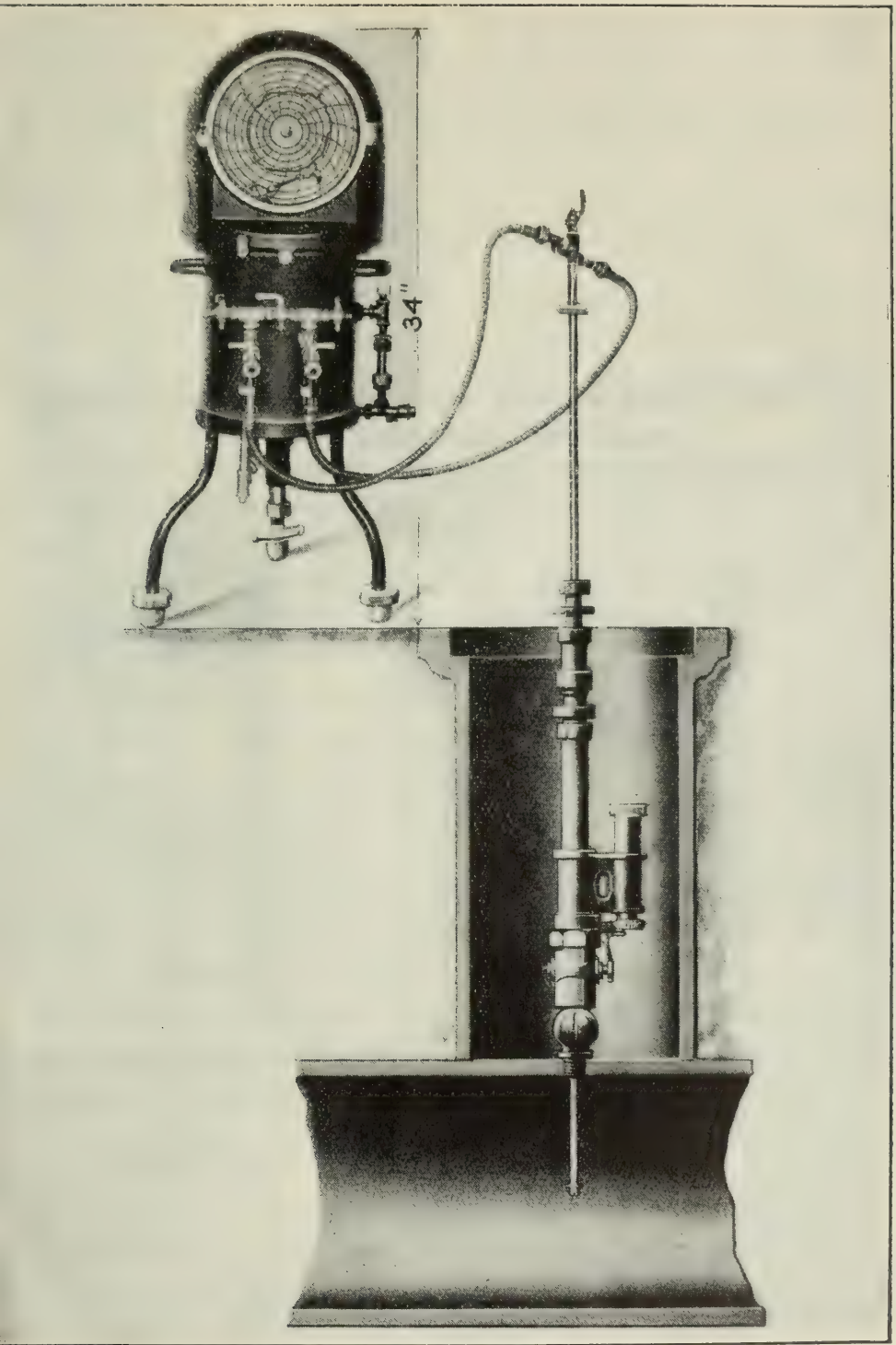


FIG. 8.—PORTABLE PITOT RECORDER. FOR MEASURING AND RECORDING FLOW OR LEAKAGE IN WATER MAINS, FLUMES, PENSTOCKS, ETC.

TABLE 3.—Test of a Recording Register Based on this Principle and with Pitot Tube Heads.

Up or down	C	V	V_1	D	P
U	0.008	0.5	0.516	+0.016	+3.2
D	0.007	0.5	0.483	−0.017	−4.0
U	0.031	1.0	1.017	+0.017	+1.7
D	0.029	1.0	1.985	−0.015	−1.5
U	0.071	1.55	1.540	+0.040	+2.65
D	0.068	1.50	1.507	+0.007	+0.45
U	0.122	2.0	2.020	+0.02	+1.00
D	0.119	2.0	1.99	−0.01	−0.50
U	0.274	3.0	3.02	+0.02	+0.66
D	0.268	3.0	2.99	−0.01	−0.33
U	0.484	4.0	4.02	+0.02	+0.5
D	0.478	4.0	3.99	−0.01	−0.25
U	0.756	5.0	5.02	+0.02	+0.40
D	0.752	5.0	5.01	+0.01	+0.42
U	1.084	6.0	6.02	+0.02	+0.33
D	1.082	6.0	6.01	+0.01	+0.17
U	1.476	7.0	7.01	+0.01	+0.14
D	1.474	7.0	7.01	+0.01	+0.14
U	1.928	8.0	8.01	+0.01	+0.13
D	1.929	8.0	8.02	+0.02	+0.25
U	2.440	9.0	9.02	+0.02	+0.22
D	2.440	9.0	9.02	+0.02	+0.22
U	2.980	10.0	10.00	0.00	0.00
D	2.980	10.0	10.00	0.00	0.00
D	0.315	1.0	1.025	+0.025	2.5

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1268

RECENT IMPROVEMENTS IN LEVELING INSTRUMENTS.*

By Dunbar D. Scott, M. Am. Soc. C. E.

While the geodesist and the precise mechanic have been striving by independent or collateral effort to raise the art of leveling to the dignity of an exact science, it is quite evident that such investigations have induced too great a complexity in instrumental construction.

In any leveling operation the tangential axis of the bubble should be regarded as the datum plane, and the line of sight as the base line on which the survey is constructed. If the telescope is properly collimated, the line of sight, or rather the plane of vision indicated by the central horizontal wire, will occupy the equator of the field of view, and, for any ordinary distance, will describe a true horizon when regulated by a properly adjusted spirit-level of sufficient sensibility.

The fundamental requirements in any leveling instrument are simple in the essentials, but they have been made intricate by the intrusion of mechanical details between the elements on which depend the accuracy and rapidity of the operation.

The primary object is to preserve parallelism between the bubble-axis and the line of sight; and to provide an easy method of making a telescopic observation simultaneously with an ordinary observation, necessary to adjust the bubble accurately to the center of its run.

In the Wye-level† we are required to test this most vital adjustment through the Y-supports and the collars, which, at various times,

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* Presented at the meeting of May 21st, 1913.

† Invented by Jonathan Sissons, of London, in 1740. See article by Daniel Eckstrom in the *Journal*, Royal Swedish Acad. of Sci., Vol. V, 1743, p. 144.

have been made of red metal, bell metal, steel, or invar, and supported on ivory or agate pivots, depending on the degree of precision sought in the work. The collars are, in reality, at the basis of adjustments, and the idea of supporting them on agate pivots, or anything else that would tend to wear them out, is only one of the fallacies experienced in the attempt to eliminate the instrumental inequalities which affect the final result.

In the Wye-level, we are also dependent, for a successful performance, on the perfect verticality of the vertical axis, which should be considered preferably only as a means to an end; for, if by some more facile method we can adjust the bubble and the sight-line to parallelism, we are certain of projecting a truly horizontal line when the bubble occupies the center of its run, irrespective of the conditions existing in the leveling base.

The practice of interrupting the connection between the telescope and the vertical axis with a set of horizontal elements pivoted at one end and connected at the other with a "vertical adjustment" through a micrometer or gradienter screw, was first utilized by George F. Brander, of Augsburg, in 1769. This constitutes the first great and only fundamental improvement that has ever been applied to the mechanical construction of the Wye-level. It was adopted by Professor Stampfer, of Vienna, then by Kern and other European manufacturers, until to-day, it is practically the only type of Wye-level used on the Continent.

The pivots were not moved to the center of the base-bars until Ertel, of Munich, did so in 1842;* and, as we learn from Englebreit's "Instrumente der Geodasie" (Nurnberg, 1852), they were soon after elevated in the cradle-wye to the center line of the telescope where no amount of correction would have any effect upon the height of instrument.†

The Stampfer, or the Kern model, which does not differ in essential details of construction, was first adopted in the United States Lake Survey in 1876, and was used to the exclusion of other types in the U. S. Coast and Geodetic Survey until 1900.

Professor Stampfer, who applied the gradienter screw of this con-

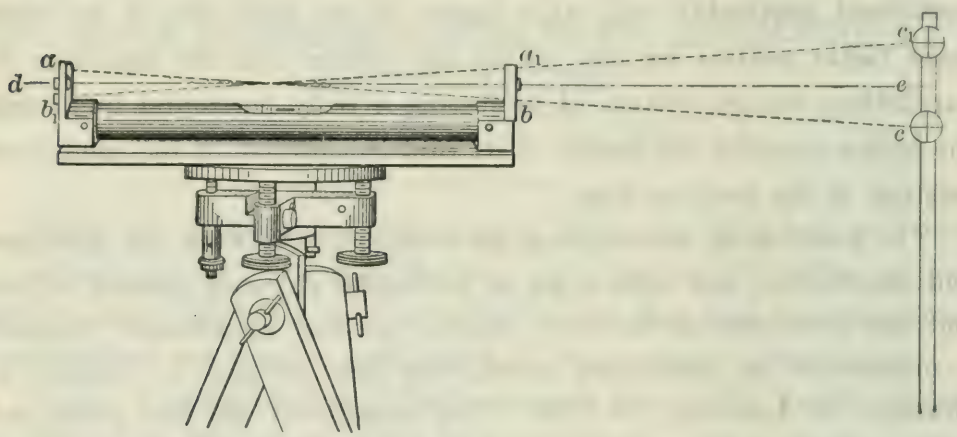
* See article by Cotta in *Dingler's Polytechnisches Journal*, Vol. 84, 1842.

† See also "Die Geomatrischen Instrumente," Dr. G. C. Hunäus, Hanover, 1864, p. 429.

struction to the portable transit instrument in the early Seventies of the last century, began experiments, in the early stages of his career, by applying to all leveling operations the principle of reversion in the widest sense, not only to the instrument itself, but to the operations performed with the instrument.

In principle, the primitive diopter of ancient origin was well adapted to the theory, as will appear from inspection of Fig. 1.

It is difficult to say where instruments of this character had their origin, but we need not look back farther than 1660 when Thevenot



ANCIENT DIOPTER,
SHOWING REVERSION PRINCIPLE
OF ADJUSTMENT

FIG. 1.

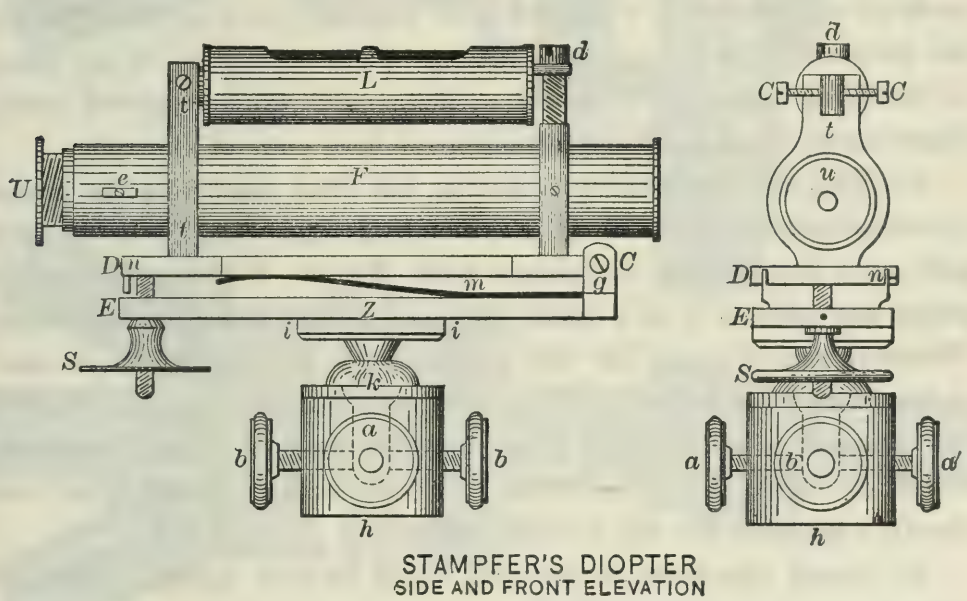
invented the bubble-vial. They were supplied only with peep-holes and crossed horse-hair sights, but their construction reduced the process of adjustments to the simplest possible terms.

In this case we are relieved of any necessity for perfect verticality in the vertical axis. If the first sight, which may be assumed as having been taken through *ab,* intersected the rod at *c,* on reversing the instrument, and re-centering the bubble, if necessary, we should have the second observation, as through b_1 a_1, intersecting the rod at c_1, under precisely the same conditions which controlled the first observation. Having established by this artificial means a true horizon for the height of instrument at *e,* it is not difficult to arrange the sights, *a* and *b,* into the alignment, *de,* which must be parallel with the axis of the bubble. This simple operation constitutes a most accurate and effective means of adjustment, and one which is free from the influence of any mechanical defects in the instrument itself.

In a telescopic sight, however, with a carefully collimated and unalterable sight line, we are confronted with a slightly different problem, in which the telescope must first be brought into coincidence with the mean of two such observations and the bubble finally readjusted to fit this condition.

Professor Stampfer's attempt to substitute an optical system which could be utilized for this peculiar purpose resulted in the model illustrated in Fig. 2.

The optical sight consisted of two plano-convex lenses of about 5 mm. aperture and about 32 mm. focal length, placed at opposite ends



STAMPFER'S DIOPTER
SIDE AND FRONT ELEVATION

FIG. 2.

of the tube, at twice their focal length apart. Cross-wires in the common focal plane were located midway between them.

The aperture was purposely reduced to that of the pupil of the eye, for, in such a combination of lenses, the focus must be practically universal, with an emergent beam consisting of nearly parallel rays. Nothing would be gained, therefore, by producing a larger image than could enter the eye.

Each lens in turn could be used either as an eye-piece or an objective, and focusing was unimportant or rarely necessary. The focal length of the "objective" and the "ocular" being equal, the instrument possessed no telescopic power and did not elevate the problem beyond the most commonplace consideration, except that it was possible to

secure a simultaneous view of the cross-hairs and a distant target, without straining the accommodative function of the eye.

Fig. 2 is reproduced from Bauernfeind,* who says, as one would translate:

"The method utilized the most rational means of instrumental adjustment and of balancing observations, and was really capable of very accurate work. The lengths of sights were strictly limited to the range of human vision, but experience has demonstrated that its error fell within 1 : 20 000."

In the first edition of Gillespie's "Surveying" (1855), mention is made of an attempt to construct such an instrument in America, but the proposition was abandoned because there seemed to be no means of supplying telescopic power to an instrument which required optical components of symmetrical formation at both ends of the tube.

The idea was temporarily forgotten, but after discussing the matter in correspondence with W. G. Raymond, M. Am. Soc. C. E., in 1904, and more recently in conference with Mr. G. N. Saegmueller, the writer has utilized it as a theme for considerable reflection and study. Whatever may be said for the achievements of the "practical man" or the precise mechanic, in the perfection of engineering instruments, this attempt to depart from the endless arguments attendant on the theory of compensating errors in all present instrumental types, was clearly a problem for the scientific optician.

An appeal was made to the best talent in both America and Germany,† and, after nearly three years of investigation in both the laboratory and field, telescopic optics of any desired power have been applied at last to the principle of reciprocal vision in the palindromic telescope herein described, which might be otherwise known as an optical Janus.

The new "Compensation Level" is shown in both elevations in Figs. 3 and 4, and in transition in Fig. 5. The writer is responsible for the details in its mechanical construction, which is, as it should be, only a means to an end—a vehicle of manipulation which does not in any appreciable manner enter into the character of the result.

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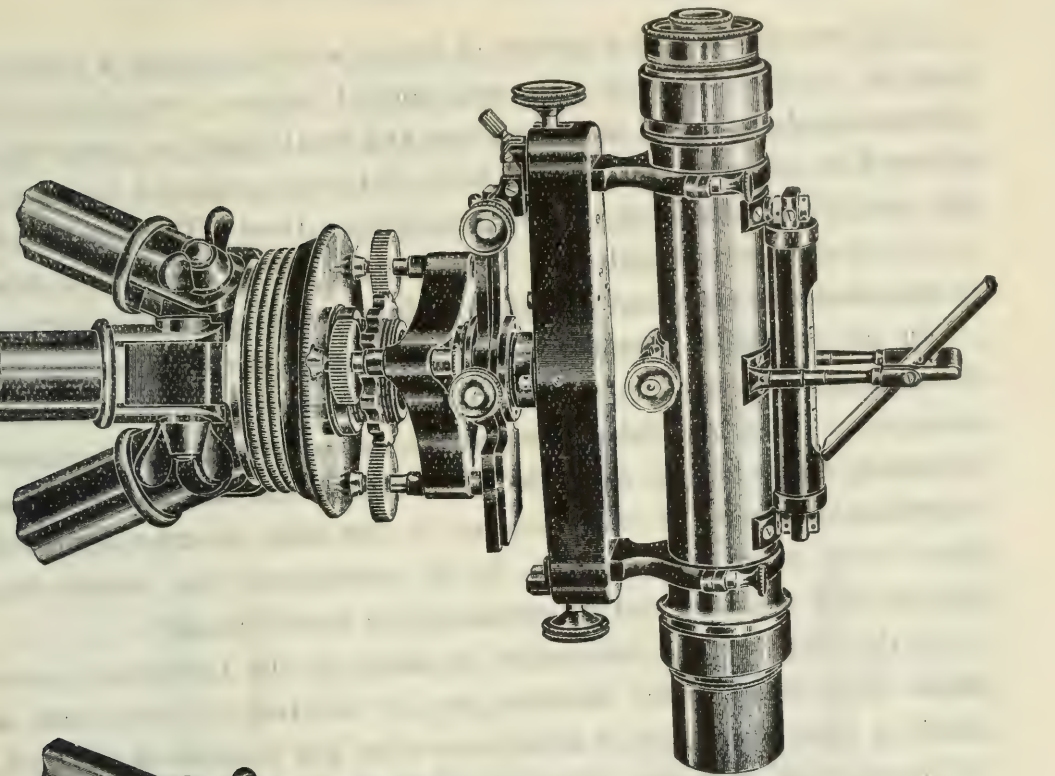
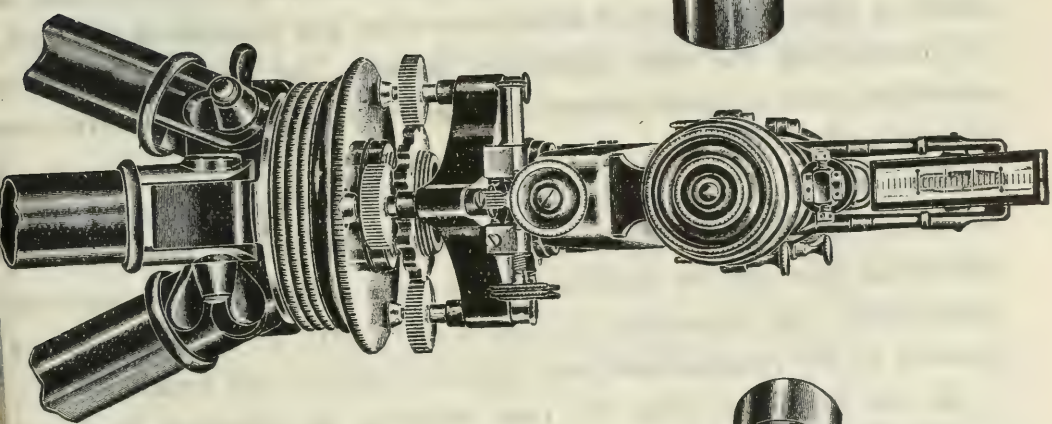
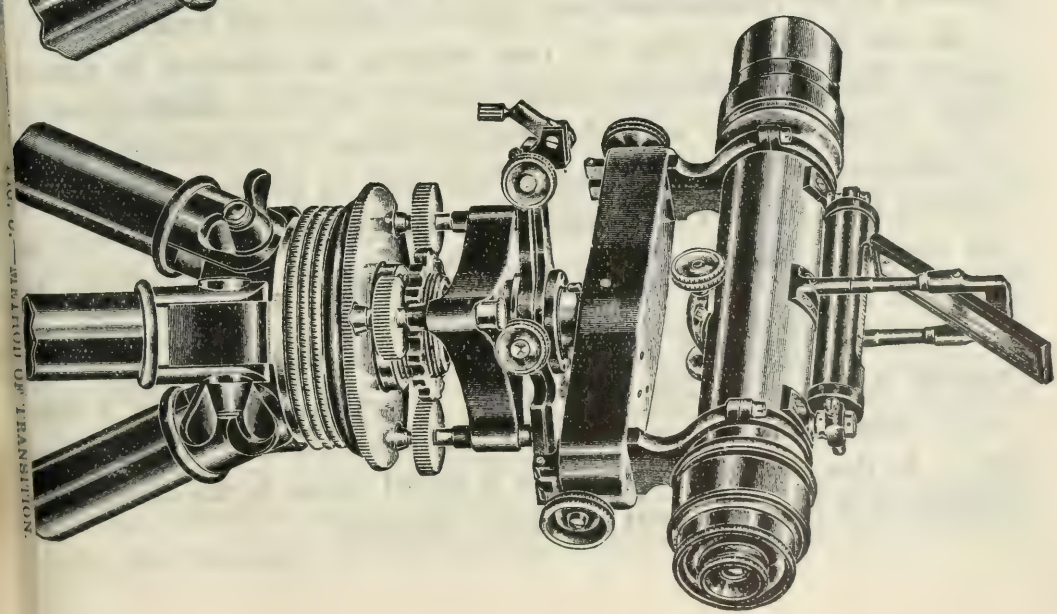
* "Elemente der Vermessungskunde," by Dr. C. M. Bauernfeind, München, 1856, Vol. 1, p. 390, etc.

† *Zeitschrift für Instrumentenkunde*, Berlin, Nov., 1909, Heft 11; *ibid.*, Vol. 12, Nov., 1892, p. 377.







This instrument has a system of optics which possesses a fascination for students of both pure and applied science, comprising two achromatic objectives of equal aperture and focal length, so immovably disposed at opposite ends of the tube that their principal foci overlap, but fall slightly short of the space between them. The lens system is a perfectly symmetrical one. What is true of one direction is true of the other; the argument, therefore, will be conducted in the singular.

In all telescopes heretofore applied to engineering instruments, focusing has been conducted either by movement in the objective draw-tube, or by the ocular tube in which the diaphragm and eye-piece are mounted; and while at least the first method was attempted in the experimental stages of this problem, it has been thought desirable, above every other consideration, to fix permanently the relationship between each objective and its corresponding cross-line sight.

If the plane of the cross-wires is placed at the farthermost conjugate focal plane of the objective, we are required to concentrate a pencil of light, the origin of which may be at any position in the field, by intercepting the rays somewhere between the objective and the cross-lines, with a double concave or negative lens ground to curvatures of long radius.

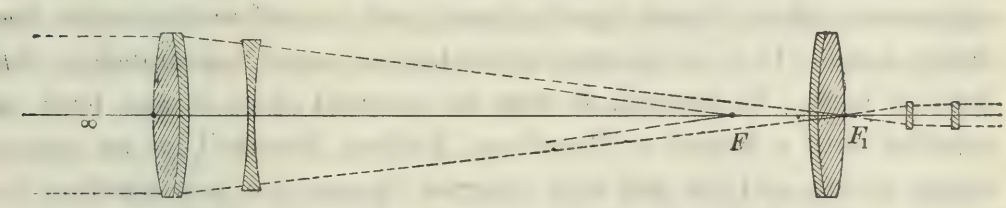
This, therefore, is a lens of low sensibility, the function of which is to divert slightly the convergent rays to suit every circumstance, depending only on the position in the tube at which these rays are intercepted. An inspection of Figs. 6 and 7 will give an idea of the position of the intermediate lens, both for infinity and proximity.

In the first case (Fig. 6), rays entering the objective in nearly parallel lines, from the most distant points in the field, would naturally form an image at, or near, the principal focus of the objective at F, as indicated by the longer dotted lines; but, in passing through the negative lens, the rays are slightly bent toward the prism base, and do not come to a focus until they reach the plane of the cross-wires, as indicated at F_1.

If the incident beam enters the objective in divergent rays from a near-by object, as in Fig. 7, it would naturally converge on a conjugate focal plane, at, or very near, the fixed position of the cross-lines. In this case the negative lens is moved nearer to the diaphragm, where it will exert the least refractive influence.

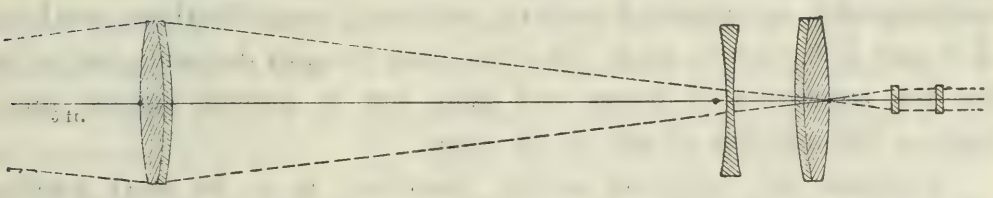
This arrangement of lenses does not preclude the use of the stadia interval, and provides a most efficient and satisfactory means of focusing, which may now be adopted in tachymeters or theodolites for mines and other such purposes, where an impervious telescope and greater security in the collimation adjustment are of prime consideration.

The intermediate negative lens is of relatively low power compared with that of the objective, and the whole system of lenses is corrected for aberrations, as a single combination, for a mean distance in the field. For very long sights, spherical and chromatic aberrations are very slightly over-corrected, and for short sights, slightly, but not noticeably, under-corrected.



POSITION OF NEGATIVE LENS FOR DISTANT OBJECTS.

FIG. 6.



POSITION OF NEGATIVE LENS FOR NEAR-BY OBJECTS.

FIG. 7.

The negative lens is also ground to curvatures in which the incident angles are very nearly constant while moving it in the optical axis. By observing these precautions, no greater effect on the quality of the image or definition is apparent, than in the ordinary method of focusing.

As applied to leveling instruments, the intermediate lens would suggest, to a highly theoretical mind, the possibility of not only subduing the illumination, but of modifying the collimation adjustment which constitutes the one and only imperative requirement on which the accuracy of this instrument depends.

The scientific optician allows for a loss of about 8% in illumination, by reflection and absorption, in any polished lens of ordinary

thickness; but this phenomenon is apparent only to a trained eye, and may be nearly corrected, without making a sacrifice to other optical qualities, by increasing the diameter of the objective 4 or 5 per cent. In this instrument, as illustrated in Figs. 3, 4, and 5, there are no more lenses than are necessary in any terrestrial telescope commonly used by the American engineer for most azimuth instruments and for many types of levels.

To favor light conditions, under these circumstances, it has been thought desirable to consider, by preference, the astronomical (inverting) ocular which is most advantageously suited for leveling instruments, whereas it might cause some annoyance in transits or theodolites.

When the target rod is used, the target itself is not changed in appearance when viewed upside down, and in self-reading rods the figures have only to be inverted to produce a normal appearance in the field of view. In general, it may be assumed that, if any level is supplied with a Ramsden, Huyghens, Kellner, Steinheil, or an orthoscopic ocular, and the rod with inverted figures, the best selection in the character of the outfit has been procured.

The optical arrangement in this instrument is such that two interchangeable astronomical oculars, producing magnifications equal to $X18$ and $X26$, may be used. This has been thought desirable, not only for the regulation of power and light, but to provide against misfortune through loss of one or the other.

Although the inverting ocular, therefore, is to be most highly recommended, it is not impossible to utilize the erecting type, as shown in connection with Fig. 10, by which a magnification of $X20$ is secured. With this eye-piece, the superlative optical qualities, with respect to illumination and size of field, cannot be expected, but it has been designed as an option for those who are confused by an inverted image. The erecting eye-piece, together with an erroneous idea that the accuracy of the sight is proportional to the length of the optical axis, are the factors which determine the unusual length of the modern Wye-level. The use of an erecting eye-piece, with the instrument under consideration as a substitute for the inverting ocular, increases the length of the telescope 40%, in-

FINE CROSS-LINES ETCHED ON SURFACE OF CROWN LENS

FIG. 8.

creases the weight of the instrument 20%, and reduces considerably the brightness of the image.

After much experimentation with a new three-ring diaphragm (which was finally discarded), it was decided, inasmuch as the objectives were immovably and permanently fixed in the tube, that each, in turn, could be made to serve as an excellent substitute for a glass diaphragm for the other optical system.

With this in view, it became necessary to design an elaborate collimating apparatus especially for this instrument. Where the optical axis of each objective passes through the other, a microscopic cross is etched on the outer surface of the crown lens, with a fine diamond point, somewhat as shown in Fig. 8. If the elements which enter into the collimation adjustment, therefore, are fixed, we may rationally assume a permanency in the adjustment itself, and look only to the interior focusing lens for such derangements as are possible under ordinary conditions of work.

The diaphragm opening in which the cross-lines appear is necessarily, in this case, equal to the full aperture of the objective. This condition makes it impossible to margin off the extreme outer rays which produce a secondary spectrum, but the field of view is much larger than that which is utilized by either of the eye-pieces, and a slight fringe of color in the periphery is not noticeable, unless a deliberate search is made. The working part of the field of view is free from spherical or chromatic aberration, and possesses all the optical qualities which satisfy the most fastidious demand. There is nothing, however, to prevent the construction of a diaphragm of proper proportions as a part of the eye-piece mount, and so fulfill in this particular the uttermost requirement in the case.

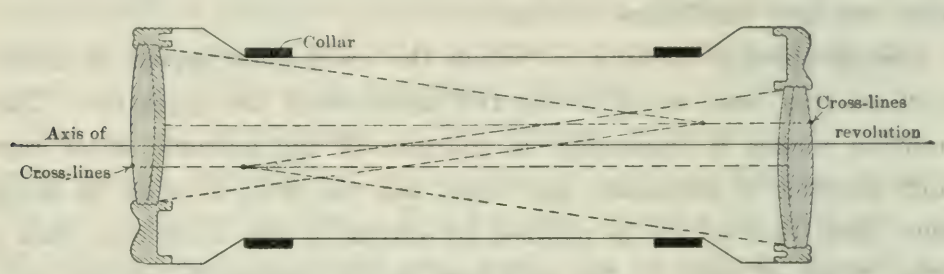
The position of the plane of the cross-lines has been located arbitrarily where an image will be naturally produced by rays having their origin 5 ft. beyond the objective. For the shortest sight possible to be observed through either end of this instrument, 1.5 m. has been selected.

Particles of dust will inevitably settle on the outer surfaces of the objectives, and will be noticeable in the plane of the cross-lines under the magnifying effect of the eye-piece. Unless one is familiar with this construction, such a phenomenon will cause either wonder or con-

fusion. The glasses should be cleaned occasionally with a camel's hair brush.

The collimation adjustment consists of the fixation of the intersection of the cross-wires into the optical axis of the objective, as accomplished by some improvised axis of longitudinal revolution. The collimation line in this, or any ordinary, telescope may be slightly removed from this geometrical axis, depending on the amount of eccentricity which exists in the objective mount; but it will be parallel in any case.

A thorough investigation into the probable causes of collimation error, in reconciling one optical system with the other, induces the conclusion that the maximum total error in reading must be equal only to the amount of eccentricity in the objectives, which is not a progressive error in any event, and, if it occurs in the horizontal plane, is a negligible quantity, however great.



EFFECT OF ECCENTRIC MOUNTING ON COLLIMATION.

FIG. 9.

With the facilities of manufacture now at the disposal of the precise mechanic, we shall perjure the premises if we allow that, in mounting, a greater displacement than 0.1 mm. may occur on either side of the geometrical axis of the tube. Treating, however, with a most exaggerated case of this character, we shall have, after the separate adjustments for collimation, the conditions expressed in the diagram, Fig. 9, showing two possible sight-lines that must be either coincident with, or parallel to, the axis of revolution.

Eccentricity of mounting will reveal itself, after the collimation adjustment, if the telescope is revolved in the wyes. If the field of view appears to move in a small circle, there is eccentricity of mounting; but, if the collimation adjustment has been properly made, the intersection of the cross-lines will appear to move with the field.

There will be no such thing as an inclination of the optical axis

in the tube by virtue of an inclination in the objective itself. No error can possibly occur through oblique mounting, except that of astigmatism, but this is incidental rather than vital, and need not be considered in this, any more than in any other, instrument.

The chances for error induced by any lateral movement in the focusing lens have to deal first with the collimation adjustment (or the position of the optical axis as fixed by the cross-lines), then with the reproduction of the rod at the plane of the cross-lines, thus fixed.

In testing the accuracy of the collimation adjustment for either optical system, some very distant point in the horizon should be selected. If the test is made on a near-by object, it will not fulfill, in the highest theoretical sense, the requirements in the case. The infallible test for errors of this character is a check of the collimation on a near-by object after perfecting the adjustment on some very distant point. This necessitates moving the interior lens between the positions shown in Figs. 6 and 7; and this test ought to qualify, irrespective of eccentricities in mounting, as previously shown by D. A. Molitor,* M. Am. Soc. C. E.

An excellent collimator apparatus for use in the field may be arranged by placing the objectives of a level and a transit in juxtaposition so that a beam of light can pass directly through both optical systems. If the transit telescope is racked down to its shortest length, it will be necessary to focus the telescope of the level at, or near, infinity, in order to see the cross-wires of the transit diaphragm. This method brings an object, apparently at an infinite distance, into very convenient range, and the test may be made at night by placing a bright light immediately behind the transit eye-piece. The arrangement, as illustrated in Fig. 10, shows the level equipped with an erecting eye-piece and a special sunshade which may be used interchangeably with the inverting eye-pieces, and thus convert the telescope almost instantly from one type to the other.

The axis of revolution is established as between the bearings in the supports. It is desirable, therefore, that the caps should be only slightly loosened and that the telescope should not be removed from its supports, or reversed for the second test, except on the vertical axis. In making the collimation test by longitudinal revolution, it is not

Transactions, Am. Soc. C. E., Vol. XLV, 1901, p. 45.

absolutely necessary that the telescope should be horizontal; but the axis of revolution in the wyes must not be modified except as stated.

The possibility of securing permanently the collimation adjustment by fixing immovably the relationship between the elements which constitute that adjustment, commends itself at once, and, now that we

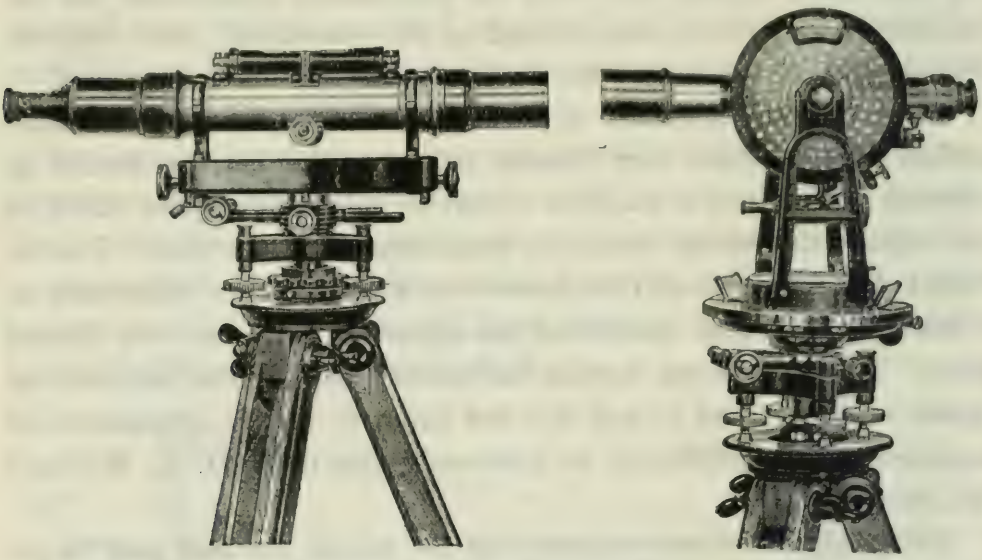
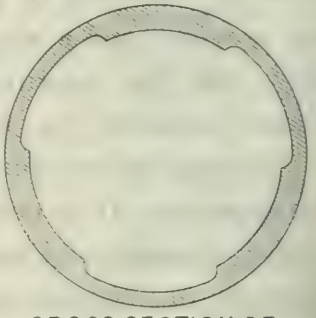


FIG. 10.—FIELD COLLIMATOR APPARATUS, SHOWING READING MIRROR REMOVED AND BEARING CLIPS LOOSENED TO FACILITATE LONGITUDINAL REVOLUTION.

may provide against any movement for either the objective or the diaphragm, we may mount them in a tube which is sealed against the intrusion of dust or moisture.

The telescope tube has been reinforced with three longitudinal ribs, cast, drawn, and turned down in one solid piece with the outer wall. The carefully machined ribs add lateral strength, prevent flexure, preserve alignment, and form a convenient skidway on which to slide the barrel in which the focusing lens is mounted. The contact surfaces between these ribs and the lens-barrel are the only movable parts on which the accuracy of the instrument depends.



CROSS-SECTION OF TELESCOPE TUBE

FIG. 11.

If we safely allow that the error due to eccentric mounting of the objectives is a constant not exceeding 0.2 mm. for all distances, it may be regarded as a negligible quantity, and the proposition seriously concerns us only

so far as irregular motion in the focusing device may affect the reproduction of the field of view at the plane of the cross-wires.

This influence has been accorded a most careful theoretical and practical investigation, with results so satisfactory that it is recommended as superior in point of accuracy to either of the other methods of focusing now in common use.

A theoretical consideration of the effect produced on the optical axis by the negative focusing lens is one involving some of the simple laws of refraction. For the purposes of this demonstration, Fig. 12 represents an exaggerated condition of affairs.

Let it be assumed that an objective, O, and a reticule, R, are immovably placed with respect to each other. It is assumed that the negative lens, being in the position, N, when focused on infinity drops

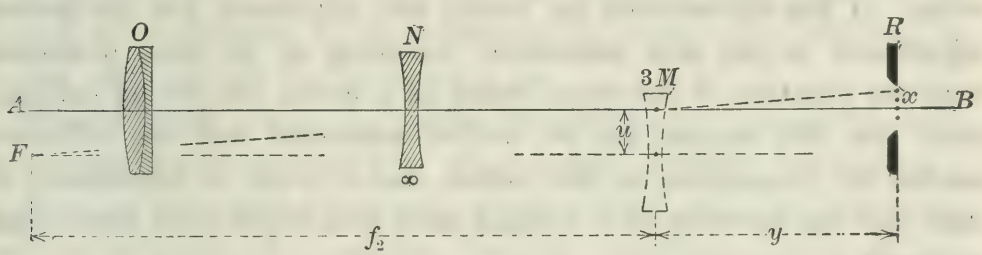


DIAGRAM SHOWING EFFECT OF LOST MOTION IN FOCUSING LENS.

FIG. 12.

out of alignment from the optical axis, AB, to the position, $3M$, through a displacement equal to u, when focused on an object, say, 10 ft. away.

In the position, $3M$, the axial ray from A will be deflected at an angle having a base equal to x, and the anterior focal point will drop accordingly to F. Let the total amount of deflection at the diaphragm be equal to x, and let the distance from F be equal to f_2. From similar triangles we have:

$$x : u : : y : f_2$$

$$x = u \frac{y}{f_2}.$$

In the very portable instrument under discussion, the focal length of the negative lens, f_2, is equal to 350 mm., and the mean value of y is equal to about 70 mm. Substituting, we have:

$$x = u \frac{70}{350} = \frac{1}{5} u.$$

In other words, for a level of this length, the displacement of the image at the diaphragm will be equal to about one-fifth of the deviation of the focusing lens from the optical axis.

The contact bearings of the inner lens-barrel can be made to move so accurately that, whether we consider the error of displacement, as applied to collimation adjustment or accuracy of pointing, we can only conclude that it must fall within the diameter of the sight line itself, and remain within the limbo of intangible quantities.

The mechanical process of focusing by this method necessitates only a movement of the interior lens between the telescope supports, where such a reapportionment of weight will not sensibly affect the center of gravity or destroy the equilibrium of the instrument.

The bubble tube is rigidly attached directly over the top of the telescope. It has adjustments for height and alignment, but the lateral adjustment is not now important, inasmuch as we have dispensed with the function of the wyes, except as a means for testing collimation. On this account we are neither concerned with the influence exerted by inequalities in the collars, due to wear or concretions of dust, nor the necessity of a striding level with which such errors may be tested.

If temperature fluctuations have any effect on the horizontality of the level vial, the elimination of such discrepancies is accomplished only by the process of end-for-end reversals, which forms a part of the prescribed routine in the performance of precise leveling with an instrument of this type.

The accuracy of results depends entirely and directly on the relation of the bubble axis to the amphidioptric sight line, and sudden changes in temperature will affect such work only as they are capable of disturbing this relationship.

H. M. Wilson, M. Am. Soc. C. E., concurs with Professor Molitor in the opinion that precise leveling does not differ materially from the ordinary kind, except in the character of the instrument; and that practically all residual errors are attributable to temperature changes.* It would seem, therefore, if we provide in the new instrument a ready means of testing the collimation, that temperature effects will compensate themselves on reversals, and that it will be no longer necessary

* *Transactions*, Am. Soc. C. E., Vol. XLV, 1901, p. 126.

to equalize forward or back-sights, unless it is thought desirable to do so on account of curvature and refraction.*

Although variations in temperature may affect somewhat the angular value of the graduations on the bubble vial, its capability to indicate a truly horizontal setting is not impaired if the same central segment of the graduated scale is used, as advocated by the late J. B. Johnson, M. Am. Soc. C. E.

At each side of the bubble tube are sockets for the reception of the mirror supports. The mirror frame is hinged at the center, so that the reflected image of the bubble may be viewed from either end of the telescope with equal facility. This method of observing the bubble was adopted on account of its peculiar fitness to the requirements of the case; but a stop has been provided that regulates the amount of inclination to approximately 40° each way, and also to prevent cracking the vial by reckless use.

Naturally, the longer the bubble the greater the amount of error, due to parallax in reading it by this method; but this constant, in a sensitive bubble, must necessarily be very small, and will neutralize itself in reversals. For ordinary leveling operations it may be disregarded entirely. It would seem, from the peculiar conditions and the requirements in the case, that by the process of end-for-end reversals every real or imaginary error, traceable to the bubble or its mounting, will be perfectly compensated and automatically corrected.

Our total anxiety, therefore, in the use of the instrument may be concentrated on the parallelism between the bubble axis and the biaxial sight-line to which it must be adjusted; but this can be tested at any time, within from 3 to 5 min., in a second observation, without moving out of one's tracks and without an assistant.

For ordinary leveling purposes, the operator will naturally make his test in the morning, as required of all assistants in the Coast Survey, and, when satisfied, he will apply the eye-piece to one objective and the sun-shade to the other, for continuous and accurate work. This instrument was designed only for use in more ordinary work, but the object of this paper is to show that the construction is such that the most precise results may still be secured by two sets of

* See L. S. Smith, M. Am. Soc. C. E., in *Bulletin*, Univ. of Wis., Vol. I, No. 5, 1895; W. S. Williams, Assoc. M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. XLV, 1901, p. 176; "Plane Surveying," Professor J. C. Tracy, 1898, p. 273, etc.

double observations, on the assumption that the collimation and the bubble are both out of adjustment.

The instrument's first claim on the attention of the engineer is the wonderful facility with which the adjustment for parallelism may be tested, and the accuracy and speed with which it may be rectified. An unbiased, open-minded examination into those parts of the instrument which might be suspected of infidelity, develops the conclusion that, one by one, they may be reduced to negligible quantities.

The vertical adjustment device is an entirely new model, peculiarly adapted to the reversible qualities of this instrument. The upper element of the base-bar forms a channel encasement for the lower element, which is rigidly attached to the vertical axis.

The telescope and bubble are mounted on the upper element in non-adjustable supports. The upper element, with telescope and bubble, revolves through short arcs in the vertical plane, on a conical pivot or an eccentric worm gear at the center of the bars, some 55 mm. below the axis of the telescope.

This is an improvement over the method of placing the pivots at one end, but nothing essential would be gained by a further modification of the mechanical equipment, in order to get the pivots in the horizontal plane which is coincident with the longitudinal axis of the telescope. The reversion principle in this level demands that we shall place the pivots at the center, in order that the height of instrument shall not be modified in the second observation. If the instrument, considered in this paper, was intended for the measurement of small vertical angles by a micrometer screw, it is agreed that the longer base secured by placing the pivots at the opposite end would contribute to the efficiency of the construction; but in the 10-in. (25-cm.), instrument herein described, the micrometer is not adaptable, and it is proposed, further, that the micrometer method of securing precise results shall be superseded by the process of reciprocal vision herein described, or by the three-wire method already adopted.

The mechanical construction of the "vertical adjustment" in this design cannot be adapted for use as a gradienter because the value of one division on the drum would change constantly in the different positions of the eccentric gear. For convenience of manipulation, we are required to communicate a slight vertical movement through a horizontal bar revolving on its own axis.

The horizontal bar terminates in milled-heads, symmetrically placed directly under each successive position of the eye-piece, so that, whether in direct or reverse position, the final delicate setting of the bubble can be accomplished without inconvenience or confusion. This method is quite unique, and is well adapted to this construction.

The upper base-bar is not always necessarily parallel with the lower, nor perpendicular to the vertical, axis. A normal position, however, may be quickly determined in reversals on the vertical axis by correcting half of the bubble displacement in the leveling base and the other half in the "vertical adjustment." In one of two such trials the bubble should remain centered during an entire revolution.

The mechanical separation of the connection between the telescope and the vertical axis distinguishes this instrument at once from the dumpy type, and places it formally in the precise class, irrespective of its numerous other claims to this distinction.

On precise levels of the Coast Survey type, a circular-box bubble has been attached to the lower element of the base-bar to secure a nearly vertical setting of the vertical axis, without regard to the relationship of the upper portion of the instrument, and independent of the more sensitive vial which is mounted on the telescope, but in the comparatively small instrument herein described a "normal adjustment" is of such easy accomplishment that it has seemed preferable to dispense with the circular bubble, which has no given sensibility or means of adjustment.

This alternative seems the more to commend itself inasmuch as the "vertical adjustment" is constructed on an endless worm gear in which the total amount of elevation or depression cannot exceed 1.8 mm. on either side of the normal line. If either milled-head were turned continuously, the telescope would only rise and fall in uninterrupted oscillation. The operator will become quickly accustomed to the effect produced in the mirror by turning the milled-head either to the right or left, and this effect will be the same when viewed from either end of the instrument.

The tangent release mechanism is also a new idea, absolutely without precedent, so far as the writer knows. As shown in Fig. 5, the nose-piece, instead of being rigidly attached to the lower bar, is removable and adapted to engage a spring-slot at either end of the instrument, as shown. In reversing the instrument, the clamp-screw to

the vertical axis need not be thrown open, as usual, but the whole clamp-and-tangent arrangement may be allowed to remain, so that it can be manipulated with the right hand in either position of the upper portion of the instrument.

When looking through either end of the telescope, the focusing screw, the vertical adjustment, the clamp screw, and the tangent screw, may be manipulated by either the right or left hand, as desired, depending on the habits of the operator, and, in fact, all other details in both the optical and mechanical equipment are symmetrical, interchangeable, and in perfect equipoise.

The type of three-screw base shown in the various illustrations is also a new model, which is attachable to the ordinary form of tripod, is applicable to the ordinary type of transit or level, and is provided

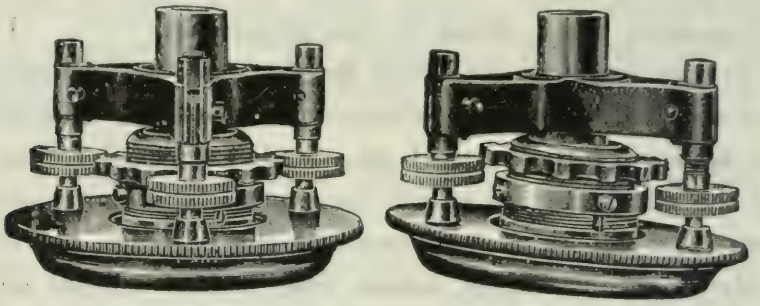


FIG. 13.—NEW THREE-SCREW BASE WITH SHIFTING CENTER.

with the regular type of shifting center such as the American engineer has used since 1858. For leveling instruments, the shifting center is not necessary, but in azimuth instruments, which are required to be centered over a given point, the appliance is almost indispensable. Referring to Fig. 13, the spring, which accommodates a universal movement through the center of the instrument, is no longer attached to an awkward spring-bar of the European model, but snugly contained in the central capsule of the shifting plate.

Every movement in each of the three leveling screws is compensated automatically at the center, and the tension in the leveling arms is equalized. Unequal strain in varying temperature, which has a subtle influence on the bubble, is eliminated, and the longer radial arms, with larger thumb-screws in German silver, provide a more sensitive control. This type of leveling base, while very convenient, is not indispensable to the instrument herein described, or to any other pro-

vided with a "vertical control," but, to the ordinary dumpy level, it adds greatly to ease of manipulation and efficiency of operation.

While in use, the coarsely-knurled clamp ring is to be loosened, but, for transportation, it is to be screwed down against the flange, in order to lock the instrument securely against the lower base.

One quickly becomes used to the three-screw system, and it is not too much to say that, if one of the screws is placed directly beneath the intersection of the two lines marking the longest axis of the bubble tubes in a transit, the bubbles may be centered simultaneously with remarkable speed.

In leveling up such an instrument as described, first secure the normal position of the bubble, then place the telescope parallel to any two of the leveling screws. After centering the bubble in this position, revolve the telescope 90° and center the bubble once more, using the third leveling screw alone.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1269

TESTS OF CREOSOTED TIMBER.

By W. B. Gregory, M. Am. Soc. C. E.

In a previous paper,* under the same title, the writer reported the tests of fourteen beams, which were taken from a trestle of a railroad near New Orleans, and, at the time of the tests, had been in use for 26 years. The specifications for treating this timber with creosote were given in the former paper, and need not be repeated here.

Recently, other beams, having been in use more than 29 years, were removed from the trestle and tested to destruction. The material was similar to that tested previously, and consisted of southern pine stringers having a cross-section approximately 6 by 16 in. and a length of 30 ft. For the purpose of testing, each beam was cut into two parts, each about 15 ft. long. It is the purpose of this paper to report the recent tests.

Method of Testing.

The tests were made exactly as in the former case, using a Riehlé 100 000-lb. machine, in the Experimental Engineering Laboratory of Tulane University of Louisiana. In order to make this discussion complete, the methods as stated in the former paper are repeated here:

"The machine is provided with a cast-iron beam for cross-bending tests. The distance between supports was 12 ft. The method of support was as follows: Each end of the beam was provided with a steel roller which rested on the cast-iron beam of the testing machine, while above the roller, and, directly under the beam tested, there was a steel plate 6 by 8 in. in area and 1 in. thick. The area was suf-

* *Transactions*, Am. Soc. C. E., Vol. LXX, p. 37.

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III
IV
V

ficiently great to distribute the load and prevent the shearing of the fibers of the wood. The head of the Riehlé machine is 10 in. wide. A plate, $\frac{3}{8}$ in. thick, 6 in. wide and 18 in. long, was placed between the head of the machine and the beam tested.

"The deflection was measured on both sides of each beam by using silk threads stretched on each side from nails driven about 2 in. above the bottom of the beam and directly over the rollers which formed the supports. From a small piece of wood, tacked to the bottom of the beam at its center and projecting at the sides, the distance to these threads was measured. These measurements were taken to the nearest hundredth of an inch. The mean of the deflections was taken as the true deflection for any load.

"In computing the various quantities shown in Table 1, the summary of results, the load has been assumed as concentrated at the center of the beam. While it is true that the load was spread over a length of about 12 in., due to the width of the head of the machine and the plate between it and the beam tested, it is also true that there were irregularities, such as bolt-holes and, in some cases, abrasions due to wear, that could not well be taken into account. Hence, it was deemed sufficiently accurate to consider the load as concentrated. Besides the horizontal bolt-holes, shown in the photographs, there were vertical bolt-holes, at intervals in all the beams. The latter were $\frac{7}{8}$ in. in diameter, and in every case they were sufficiently removed from the center of the length of the beam to allow the maximum moment at the reduced section to be relatively less than that at the center of the beam. For this reason, no correction was made for these holes. The broken beams often showed that rupture started at, or was influenced by, some of the holes, especially the horizontal ones."

In the previous paper the writer made some comparisons of results with those of large timbers tested elsewhere. The conclusion was:

"The number of tests was not sufficient to settle questions of average strength or other qualities. It will be seen, however, that the treated timber 26 years old compares favorably with the new untreated timber."

Although the tests now reported give average results slightly lower than those in the former paper, the maximum breaking load occurred in the recent tests, and the minimum of all breaking loads was found in the first set of tests. In view of these facts, it appears that the conclusions apply to the later tests of this material, which, as stated previously, has been in use more than 29 years.

The writer is indebted to his colleague, Professor J. M. Robert, of Tulane University, for valuable assistance in this work.

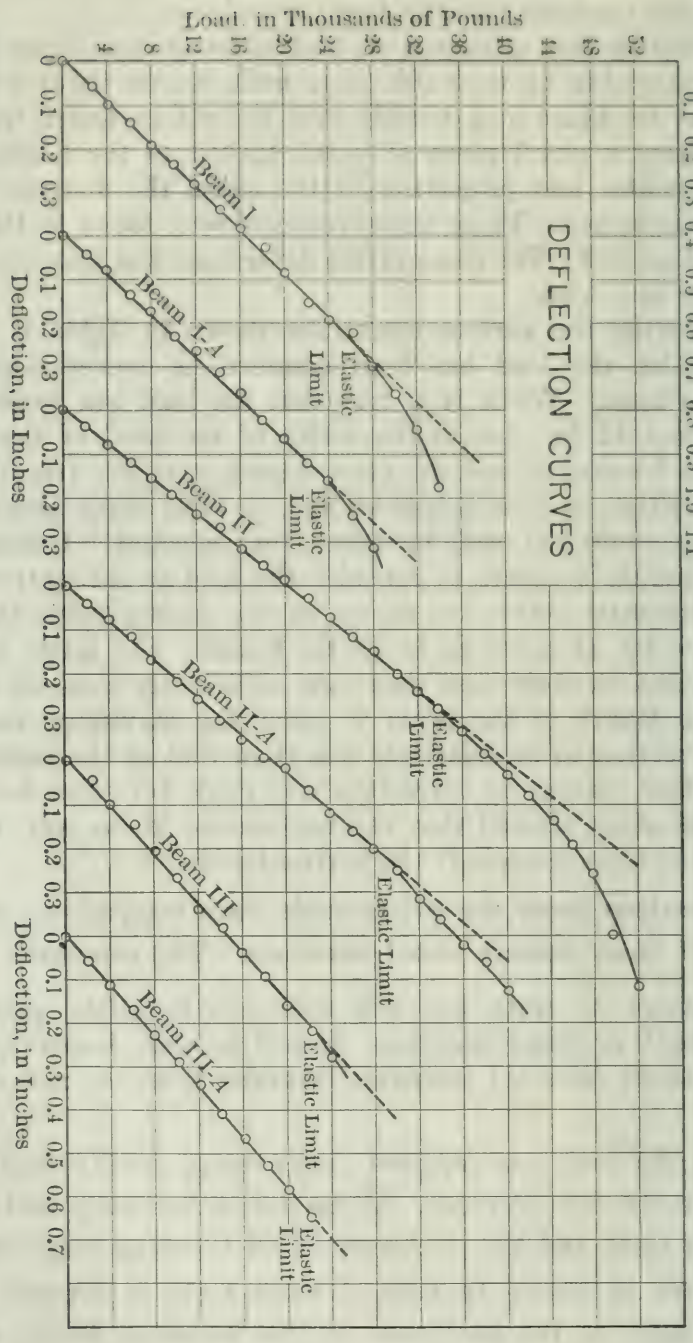
DEFLECTION CURVES
Load, in Thousands of Pounds
Deflection, in Inches
Beam I
Beam I-A
Beam II
Beam II-A
Beam III
Beam III-A
Elastic Limit

FIG. 2.

TABLE 1.—Load and Deflection Log; Beam I. Loblolly, 5 Rings per Inch.

Beam I.
Date: November 8th, 1912.
l = 12 ft.; b (mean) = 6.03 in.;
h (mean) = 15.89 in.;
c = 7.945 in. Time = ...

Beam I-A.
Date: November 6th, 1912.
l = 12 ft.; b (mean) = 6.22 in.;
h (mean) = 16.00 in.;
c = 8.00 in. Time = 32 min.

No.	P	Deflection, in Inches.				
	Load, in pounds.	Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.
1	0	1.08	0	1.12	0	0
2	2 000	1.14	0.06	1.18	0.06	0.06
3	4 000	1.18	0.10	1.22	0.10	0.10
4	6 000	1.22	0.14	1.26	0.14	0.14
5	8 000	1.28	0.20	1.31	0.19	0.195
6	10 000	1.32	0.24	1.36	0.24	0.24
7	12 000	1.38	0.30	1.40	0.28	0.29
8	14 000	1.42	0.34	1.46	0.34	0.34
9	16 000	1.47	0.39	1.50	0.38	0.385
10	18 000	1.52	0.44	1.54	0.42	0.43
11	20 000	1.57	0.49	1.60	0.48	0.485
12	22 000	1.64	0.56	1.66	0.44	0.55
13	24 000	1.67	0.59	1.71	0.59	0.59
14	26 000	1.71	0.63	1.74	0.62	0.625
15	28 000	1.78	0.70	1.82	0.70	0.70
16	30 000	1.84	0.76	1.88	0.76	0.76
17	32 000	1.93	0.85	1.96	0.84	0.845
18	34 000	1.95	0.87	2.20	1.08	0.975
19	34 350	Break.				
20	36 000	2.19		2.46		
21	38 000	2.34		2.62		
22	38 450	Maximum load.				

32 000 lb., First Crack; 38 450 lb., Failed.

At Elastic Limit: Load, 26 000 lb.; deflection, 0.625 in.; S, 3 680 lb.
Maximum: Load, 38 450 lb.; deflection, ...: S, 5 450 lb.
E = 1 283 000 lb.

P	Deflection, in Inches.				
Load, in pounds.	Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.
0	1.15	0	1.20	0	0
2 000	1.20	0.05	1.24	0.04	0.045
4 000	1.24	0.09	1.28	0.08	0.085
6 000	1.29	0.14	1.33	0.13	0.135
8 000	1.34	0.19	1.37	0.17	0.18
10 000	1.39	0.24	1.42	0.22	0.23
12 000	1.43	0.28	1.46	0.26	0.27
14 000	1.47	0.32	1.51	0.31	0.315
16 000	1.52	0.37	1.56	0.36	0.365
18 000	1.58	0.43	1.61	0.41	0.42
20 000	1.63	0.48	1.66	0.46	0.47
22 000	1.68	0.53	1.71	0.51	0.52
24 000	1.73	0.58	1.75	0.55	0.565
26 000	1.80	0.65	1.83	0.63	0.64
28 000	1.86	0.71	1.90	0.70	0.705
29 800	Broke.				

.....lb., First Crack; 29 800 lb., Failed.

At Elastic Limit: Load, 24 000 lb.; deflection, 0.565 in.; S, 3 252 lb.
Maximum: Load, 29 800 lb.; deflection, ...: S, 4 038 lb.
E = 1 243 000 lb.

TABLE 2.—Load and Deflection Log; Beam II. Long Leaf, 11 Rings per Inch.

Beam II.
Date: November 15th, 1912.
l = 12 ft.; b (mean) = 6.08 in.; h (mean) = 15.75 in.; c = 7.875 in. Time = 10.10–11.20 A.M.

Beam II-A.
Date: November 13th, 1912.
l = 12 ft.; b (mean) = 5.97 in.; h (mean) = 15.97 in.; c = 7.985 in. Time = 40 min.

No.	P	Deflection, in Inches.					P	Deflection, in Inches.				
	Load, in pounds.	Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.	Load, in pounds.	Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.
1	0	1.17	0	1.07	0	0	0	0.80	0	1.00	0	0
2	2 000	1.21	0.04	1.10	0.03	0.035	2 000	0.84	0.04	1.04	0.04	0.04
	4 000	1.26	0.09	1.14	0.07	0.08	4 000	0.88	0.08	1.08	0.08	0.08
	6 000	1.30	0.13	1.18	0.11	0.12	6 000	0.92	0.12	1.12	0.12	0.12
	8 000	1.34	0.17	1.21	0.14	0.155	8 000	0.96	0.16	1.18	0.18	0.17
	10 000	1.39	0.22	1.24	0.17	0.195	10 000	1.00	0.20	1.24	0.24	0.22
	12 000	1.42	0.25	1.29	0.22	0.235	12 000	1.04	0.24	1.28	0.28	0.26
8	14 000	1.46	0.29	1.33	0.26	0.275	14 000	1.07	0.27	1.34	0.34	0.305
9	16 000	1.50	0.33	1.37	0.30	0.315	16 000	1.12	0.32	1.38	0.38	0.35
10	18 000	1.54	0.37	1.41	0.34	0.355	18 000	1.16	0.36	1.42	0.42	0.39
11	20 000	1.58	0.41	1.44	0.37	0.390	20 000	1.18	0.38	1.46	0.46	0.42
12	22 000	1.62	0.45	1.48	0.41	0.430	22 000	1.22	0.42	1.52	0.52	0.47
13	24 000	1.66	0.49	1.53	0.46	0.475	24 000	1.26	0.46	1.58	0.58	0.52
14	26 000	1.70	0.53	1.58	0.51	0.520	26 000	1.30	0.50	1.62	0.62	0.56
15	28 000	1.74	0.57	1.61	0.54	0.555	28 000	1.34	0.54	1.66	0.66	0.60
16	30 000	1.78	0.61	1.66	0.59	0.600	30 000	1.38	0.58	1.72	0.72	0.65
17	32 000	1.82	0.65	1.70	0.63	0.640	32 000	1.41	0.61	1.82	0.82	0.715
18	34 000	1.87	0.70	1.73	0.66	0.680	34 000	1.46	0.66	1.86	0.86	0.76
19	36 000	1.91	0.74	1.80	0.73	0.735	36 000	1.50	0.70	1.94	0.94	0.82
20	38 000	1.96	0.79	1.85	0.78	0.785	38 000	1.54	0.74	1.98	0.98	0.86
21	40 000	2.00	0.83	1.90	0.83	0.830	40 000	1.58	0.78	2.06	1.06	0.92
22	42 000	2.06	0.89	1.95	0.88	0.885	42 000					
23	44 000	2.11	0.94	2.00	0.93	0.935						
24	46 000	2.17	1.00	2.06	0.99	0.995						
25	48 000	2.24	1.07	2.12	1.05	1.060						
26	50 000	2.34	1.17	2.30	1.23	1.200						
27	52 000	2.44	1.27	2.43	1.36	1.315						
28	54 000											
29	56 000											

47 900 lb., First Crack; 55 060 lb., Failed.

At Elastic Limit: Load, 34 000 lb.; deflection, 0.680 in.; S, 4 870 lb.
Maximum: Load, 55 060 lb.; deflection, ...; S, 7 890 lb.
E = 1 573 000 lb.
Sheared along neutral axis.

31 500 lb., First Crack; 42 000 lb., Failed.

At Elastic Limit: Load, 30 000 lb.; deflection, 0.650 in.; S, 4 260 lb.
Maximum: Load, 42 000 lb.; deflection, ...; S, 5 960 lb.
E = 1 418 000 lb.

TABLE 3.—Load and Deflection Log; Beam III. Loblolly.

Beam III.
Date: November 20th, 1912.
l = 12 ft.; b (mean) = 5.86 in.; h (mean) = 15.50 in.;
c = 7.75 in. Time = 30 min.

No.	P Load, in pounds.	Deflection, in Inches. Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.
1	0	1.24	0	1.19	0	0
2	2 000	1.29	0.05	1.24	0.05	0.05
3	4 000	1.33	0.09	1.30	0.11	0.10
4	6 000	1.38	0.14	1.34	0.15	0.145
5	8 000	1.43	0.19	1.41	0.22	0.205
6	10 000	1.49	0.25	1.48	0.29	0.270
7	12 000	1.58	0.34	1.53	0.34	0.340
8	14 000	1.59	0.35	1.60	0.41	0.380
9	16 000	1.65	0.41	1.65	0.46	0.435
10	18 000	1.71	0.47	1.70	0.51	0.490
11	20 000	1.77	0.53	1.78	0.59	0.560
12	22 000	1.83	0.59	1.83	0.64	0.615
13	24 000	1.89	0.65	1.90	0.71	0.680

21 500 lb., First Crack; 25 700 lb., Failed.

At Elastic Limit: Load, 22 000 lb.; deflection, 0.615 in.; S, 3 380 lb.
Maximum: Load, 25 700 lb.; deflection.....; S, 3 945 lb.

E = 1 224 000 lb.

Broke at a point 18 in. from center; bottom fibers failed in tension at bolt-hole ⅞ in. in diameter; axis of hole vertical.

Beam III-A.
Date: November 16th, 1912.
l = 12 ft.; b (mean) = 6.03 in.; h (mean) = 15.55 in.;
c = 7.775 in. Time = ...

P Load, in pounds.	Deflection, in Inches. Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.
0	0.96	0	0.97	0	0
200	1.01	0.05	1.03	0.06	0.055
4 000	1.07	0.11	1.08	0.11	0.11
6 000	1.12	0.16	1.14	0.17	0.165
8 000	1.18	0.22	1.20	0.23	0.225
10 000	1.24	0.28	1.26	0.29	0.285
12 000	1.29	0.33	1.32	0.35	0.34
14 000	1.36	0.40	1.38	0.41	0.405
16 000	1.42	0.46	1.44	0.47	0.465
18 000	1.48	0.52	1.50	0.53	0.525
20 000	1.54	0.58	1.56	0.59	0.585
22 000	1.60	0.64	1.62	0.65	0.645
22 600	Broke.				

22 000 lb., First Crack; 22 600 lb., Failed.

At Elastic Limit: Load, 22 000 lb.; deflection, 0.645 in.; S, 3 260 lb.
Maximum: Load, 22 600 lb.; deflection, ...; S, 3 350 lb.

E = 1 124 000 lb.

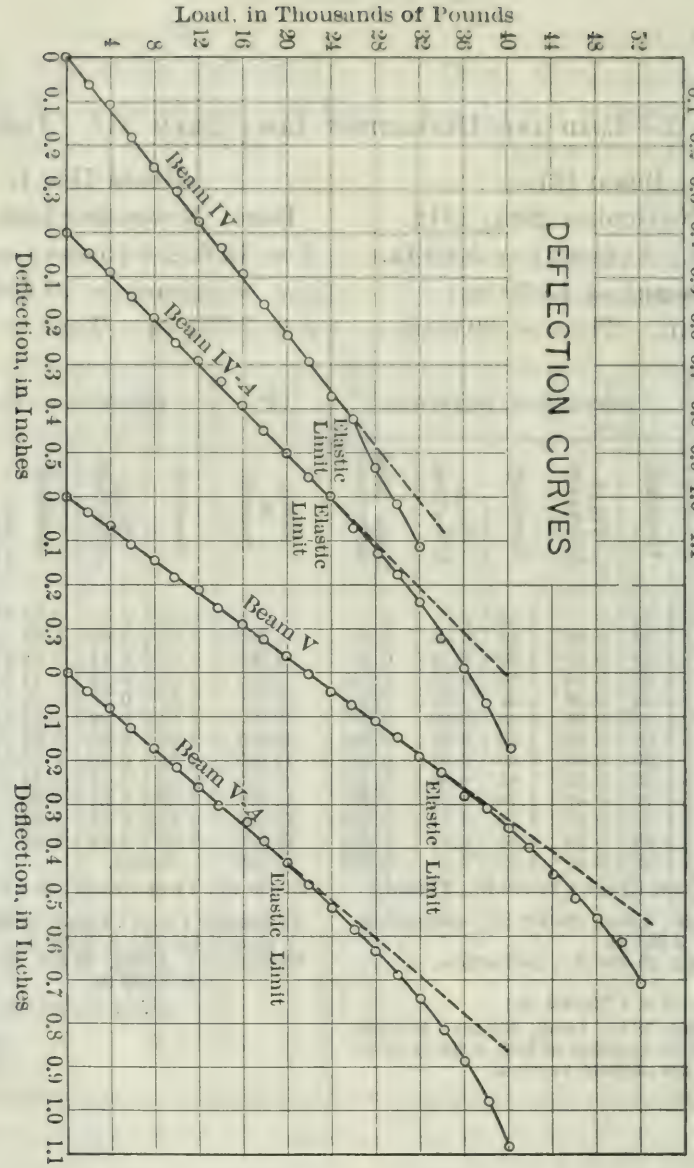
DEFLECTION CURVES
Load, in Thousands of Pounds
Deflection, in Inches
Beam IV
Beam IV-A
Beam V
Beam V-A
Elastic Limit
Elastic Limit
Elastic Limit
Elastic Limit

FIG. 3.

TABLE 4.—Load and Deflection Log; Beam IV. Long Leaf.

Beam IV.
Date: November 27th, 1912.
l = 12 ft.; b (mean) = 6.46 in.; h (mean) = 15.71 in.;
c = 7.855 in. Time = 1 hour 15 min.

No.	P Load, in pounds.	Deflection, in Inches. Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.
1	0	1.16	0	1.24	0	0
2	2 000	1.23	0.07	1.30	0.06	0.065
3	4 000	1.28	0.12	1.34	0.10	0.110
4	6 000	1.36	0.20	1.41	0.17	0.185
5	8 000	1.41	0.25	1.49	0.25	0.250
6	10 000	1.47	0.31	1.54	0.30	0.305
7	12 000	1.52	0.36	1.62	0.38	0.370
8	14 000	1.59	0.43	1.68	0.44	0.435
9	16 000	1.65	0.49	1.74	0.50	0.495
10	18 000	1.73	0.57	1.80	0.56	0.565
11	20 000	1.78	0.62	1.88	0.64	0.630
12	22 000	1.85	0.69	1.94	0.70	0.695
13	24 000	1.92	0.76	2.02	0.78	0.770
14	26 000	1.97	0.81	2.08	0.84	0.825
15	28 000	2.09	0.93	2.18	0.94	0.935
16	30 000	2.17	1.01	2.26	1.02	1.015
17	32 000	2.26	1.10	2.36	1.12	1.110
18	34 000					

31 500 lb., First Crack; 33 740 lb., Failed.

At Elastic Limit: Load, 26 000 lb.; deflection, 0.825 in.; S, 3 530 lb.

Maximum Load: 33 740 lb.; deflection, ...; S, 4 575 lb.

E = 939 000 lb.

Bottom fibers broke in tension near center of specimen.

Beam IV-A.
Date: November 22d, 1912.
l = 12 ft.; b (mean) = 6.50 in.; h (mean) = 16.12 in.;
c = 8.06 in. Time = 35 min.

P Load, in pounds.	Deflection, in Inches. Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.
0	1.08	0	1.21	0	0
2 000	1.13	0.05	1.26	0.05	0.05
4 000	1.17	0.09	1.31	0.10	0.095
6 000	1.22	0.14	1.36	0.15	0.145
8 000	1.27	0.19	1.41	0.20	0.195
10 000	1.33	0.25	1.46	0.25	0.25
12 000	1.37	0.29	1.51	0.30	0.295
14 000	1.42	0.34	1.56	0.35	0.345
16 000	1.47	0.39	1.61	0.40	0.395
18 000	1.52	0.44	1.67	0.46	0.45
20 000	1.57	0.49	1.72	0.51	0.50
22 000	1.62	0.54	1.78	0.57	0.555
24 000	1.67	0.59	1.82	0.61	0.60
26 000	1.72	0.64	1.91	0.70	0.67
28 000	1.78	0.70	1.95	0.74	0.72
30 000	1.84	0.76	2.01	0.80	0.78
32 000	1.89	0.81	2.08	0.87	0.84
34 000	1.97	0.89	2.16	0.95	0.92
36 000	2.02	0.94	2.24	1.03	0.985
38 000	2.11	1.03	2.32	1.11	1.07
40 000	2.20	1.12	2.43	1.22	1.17
42 000					

38 250 lb., First Crack; 40 130 lb., Failed.

At Elastic Limit: Load, 24 000 lb.; deflection, 0.600 in.; S, 3 065 lb.

Maximum: Load, 40 130 lb.; deflection, ...; S, 5 120 lb.

E = 1 095 000 lb.

TABLE 5.—Load and Deflection Log; Beam V. Loblolly.

Beam V.	Beam V-*A*.
Date: November 29th, 1912.	Date: December 6th, 1912.
l = 12 ft.; b (mean) = 6.09 in.;	l = 12 ft.; b (mean) = 6.25 in.;
h (mean) = 15.93 in.;	h (mean) = 15.94 in.;
c = 7.965 in. Time = ...	c = 7.97 in. Time = 33 min.

No.	*P*	Deflection, in Inches.					*P*	Deflection, in Inches.				
	Load, in pounds.	Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.	Load, in pounds.	Reading.	Total deflection.	Reading.	Total deflection.	Mean total deflection.
1	0	1.085	0	0.95	0	0	0	0.98	0	0.92	0	0
2	2 000	1.11	0.03	0.99	0.4	0.035	2 000	1.02	0.04	0.96	0.04	0.04
3	4 000	1.14	0.06	1.02	0.7	0.065	4 000	1.06	0.08	1.00	0.08	0.08
4	6 000	1.18	0.10	1.06	0.11	0.105	6 000	1.10	0.12	1.05	0.13	0.125
5	8 000	1.22	0.14	1.10	0.15	0.145	8 000	1.14	0.16	1.10	0.18	0.17
6	10 000	1.26	0.18	1.14	0.19	0.185	10 000	1.18	0.20	1.14	0.22	0.21
7	12 000	1.29	0.21	1.16	0.21	0.210	12 000	1.23	0.25	1.18	0.26	0.255
8	14 000	1.33	0.25	1.20	0.25	0.250	14 000	1.28	0.30	1.22	0.30	0.30
9	16 000	1.37	0.29	1.24	0.29	0.290	16 000	1.32	0.34	1.26	0.34	0.34
10	18 000	1.41	0.33	1.27	0.32	0.325	18 000	1.36	0.38	1.30	0.38	0.38
11	20 000	1.44	0.36	1.31	0.36	0.360	20 000	1.41	0.43	1.35	0.43	0.43
12	22 000	1.48	0.40	1.35	0.40	0.400	22 000	1.46	0.48	1.40	0.48	0.48
13	24 000	1.52	0.44	1.39	0.44	0.440	24 000	1.51	0.53	1.45	0.53	0.53
14	26 000	1.55	0.47	1.42	0.47	0.470	26 000	1.56	0.58	1.50	0.58	0.58
15	28 000	1.60	0.52	1.45	0.50	0.510	28 000	1.61	0.63	1.55	0.63	0.63
16	30 000	1.63	0.55	1.49	0.54	0.545	30 000	1.67	0.69	1.60	0.68	0.685
17	32 000	1.68	0.60	1.53	0.58	0.590	32 000	1.73	0.75	1.65	0.73	0.74
18	34 000	1.72	0.64	1.56	0.61	0.625	34 000	1.80	0.82	1.72	0.80	0.81
19	36 000	1.77	0.69	1.61	0.66	0.675	36 000	1.88	0.90	1.80	0.88	0.89
20	38 000	1.79	0.71	1.65	0.70	0.705	38 000	1.98	1.00	1.87	0.95	0.975
21	40 000	1.84	0.76	1.69	0.74	0.750	40 000	2.09	1.11	1.97	1.05	1.08
22	42 000	1.88	0.80	1.75	0.80	0.800	41 275	Broke.				
23	44 000	1.94	0.86	1.80	0.85	0.855						
24	46 000	2.00	0.92	1.85	0.90	0.91						
25	48 000	2.05	0.97	1.90	0.95	0.960						
26	50 000	2.10	1.02	1.95	1.00	1.01						
27	52 000	2.20	1.12	2.05	1.10	1.11						
28	54 000	Broke.										

16 000 lb., First Crack ; 54 000 lb., Failed.

At Elastic Limit : Load, 38 000 lb.; deflection. 0.705 in.; *S*, 5 310 lb.

Maximum : Load, 54 000 lb.; deflection, ...; *S*, 7 550 lb.

E = 1 633 000 lb.

32 000 lb., First Crack ; 41 275 lb., Failed.

At Elastic Limit: Load, 22 000 lb.; deflection, 0.480 in.; *S*, 2 990 lb.

Maximum : Load, 41 275 lb.; deflection. .....; *S*, 5 620 lb.

E = 1 350 000 lb.

TABLE 6.—SUMMARY OF TESTS.

Number of beam.	Top of butt of log.	b Width, in inches.	h Height, in inches.	$I = \frac{bh^3}{12}$	Loads. Actual, at elastic limit.	Loads. Maximum.	$S = \frac{Plc}{4I}$ At elastic limit.	$S = \frac{Plc}{4I}$ Maximum.	d, in inches. At elastic limit.	$E = \frac{Pl^3}{48dI}$	Weight, in pounds per cubic foot.	Kind.
I....	*T*	6.03	15.89	2 015	26 000	38 450	3 680	5 450	0.625	1 283 000	43.3	Coarse-grained loblolly.
I-*A*..	*B*	6.22	16.00	2 125	24 000	29 800	3 252	4 038	0.565	1 243 000	42.6	
II....	*T*	6.08	15.75	1 978	34 000	55 060	4 870	7 890	0.680	1 573 000	43.7	Long-leaf pine.
II-*A*.	*B*	5.97	15.97	2 025	30 000	42 000	4 260	5 960	0.650	1 418 000	47.6	
III..	*T*	5.86	15.50	1 818	22 000	25 700	3 380	3 945	0.615	1 224 000	39.4	Coarse-grained loblolly.
III-*A*	*B*	6.03	15.55	1 889	22 000	22 600	3 260	3 350	0.645	1 124 000	39.6	
IV...	*T*	6.46	15.61	2 086	26 000	33 740	3 530	4 575	0.825	939 000	41.9	Long-leaf pine.
IV-*A*	*B*	6.50	16.12	2 271	24 000	40 130	3 065	5 120	0.600	1 095 000	45.5	
V....	*B*	6.09	15.93	2 051	38 000	54 000	5 310	7 550	0.705	1 633 000	45.1	Coarse-grained loblolly.
V-*A*..	*T*	6.25	15.94	2 109	22 000	41 275	2 990	5 620	0.480	1 350 000	42.5	

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1270

IRRIGATION AND RIVER CONTROL IN THE COLORADO RIVER DELTA.*

By H. T. Cory, M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Le Conte, Morris Knowles, Francis L. Sellew, Elwood Mead, W. W. Follett, R. H. Forbes, R. S. Buck, F. T. Robson, Andrew M. Chaffey, C. E. Grunsky, Clarence K. Clarke, U. S. Marshall, and H. T. Cory.

From almost every point of view, the Lower Colorado River, and particularly the Colorado Delta, is extremely interesting. Ever since its examination and description by members of Lieut. Williamson's exploration party in 1850, the various features, geological, geographical, anthropological, engineering, and otherwise, have been written about. In 1905 the diversion of the Colorado River into the Salton Sea and the events which followed it were so spectacular as to result in world-wide notoriety.

While engaged in re-diverting the river, the writer became impressed with the fact that the experience and information obtained should be made available to the Engineering Profession, and since then he has constantly been gathering data to that end. In February, 1907, a general paper on the subject† was contributed to this Society by C. E. Grunsky, M. Am. Soc. C. E., then Consulting Engineer to the Secretary of the Interior in United States Reclamation Service matters; so that, before giving detailed information, it seemed best to wait until time should have revealed the strong and weak points

* Presented at the meeting of January 8th, 1913.

† "The Lower Colorado River and the Salton Basin," *Transactions*, Am. Soc. C. E., Vol. LIX, p. 1.

of construction and methods. Since then, experience with the control of the Lower Colorado River, and as local executive head of the immense irrigation project of the Imperial Valley, has brought the conclusion that the various possible vicissitudes of irrigation enterprises in the United States have been so well exemplified in the region as to justify setting forth such experience in considerable detail.

Ordinarily, more information is secured from failure than from success; consequently, no apology should be due for pointing out failures as well as successes in a paper, the functions of which are primarily to furnish useful engineering information.

The Colorado River.

The United States Geological Survey has observed the discharge of the Colorado and its several tributaries since 1895, and the results are to be found in its Annual Reports and later in the Water Supply and Irrigation Papers, especially Nos. 249 and 269, on the Colorado River Basin. At various times 169 gauging stations have been maintained, and there are 76 at present.

General Discharge Characteristics.—From the data obtained at these stations, the discharge characteristics of the tributaries and main Colorado River are pretty well determined. The discharge records of the Green River, at Green River, Utah, the lowest gauging station above its mouth, and where the drainage area above it is 38 200 sq. miles, indicate a maximum flow of about 75 000 sec-ft., a minimum flow of about 700 sec-ft., and an average annual run-off of about 5 000 000 acre-ft. The greatest discharge is in June, averaging about 1 600 000 acre-ft.; the annual rise starts about April 1st, reaches its peak in the middle of June, and has passed by August 1st.

The data obtained on the Grand River indicate a proportionately great run-off and very much the same distribution throughout the year. The records, taken at Turley, N. Mex., on the San Juan River until December, 1908, and since then at Blanco, indicate an ordinary flood maximum of about 15 000 sec-ft., a minimum of 75 sec-ft., and an average annual discharge of 1 000 000 acre-ft., but with a much longer period of summer flood than in the Green and Grand.

The maximum flood discharge of the Little Colorado when it enters the Colorado River is not known, but is probably about 50 000 sec-ft. The floods are short and violent, and carry large quantities of silt in

suspension, in which regard the stream is similar to the Gila and Salt Rivers.

The Gila at Yuma is often dry, and has a maximum flashy flood discharge of probably 185 000 sec-ft. with a total average annual run-off of 2 750 000 acre-ft. Flashy floods have been known to occur in every month of the year except May, June, and July, at which times the Colorado has its maximum flow.

Power.—Excellent reservoir sites have been found on the headwaters and along the main channels of the various tributaries, by utilizing which a considerable portion of the flow could be stored for power and irrigation. Such storage would equalize the discharge, that for power having the greater relative influence. There are at present no water-power plants of any importance whatever in the whole drainage area of the Green River. A total of approximately 40 000 h.p. has been developed in the Grand, 7 000 in the San Juan, and 20 000 in the Gila Basin, in connection with irrigation construction. No data seem to be available as to the amount of energy which it is commercially practicable to develop under existing conditions on these various streams—it is obvious that there must be a vast difference between the figures for theoretically possible and for commercially feasible developments.

Irrigation.—The water of the streams making up the Colorado is already utilized for irrigation to a considerable extent. The oldest and largest development in the basin is perhaps that on the upper Green River, in Wyoming. Recently, large irrigation systems have been constructed in the Duchesne River Basin, and there is considerable irrigation around Vernal, and also Green River, Utah. Along the White and Yampa Rivers, in Colorado, meadow irrigation is extensively practiced, and projects are on foot for the irrigation of from 200 000 to 300 000 acres in this section.

Similarly, in the Grand Basin, there are extensive meadow lands in the upper part, and a half dozen small projects in contemplation for the Middle Basin which together would irrigate about 35 000 acres. In the Lower Basin is the Grand Valley Project, covering an irrigable area of 70 000 acres, and the Uncompahgre Valley Project, which, when completed, will irrigate about 150 000 acres, both by the United States Reclamation Service. Under other schemes, from 40 000 to 50 000 acres more will be irrigated.

MAP
OF THE
COLORADO RIVER
WATER-SHED
SCALE OF MILES
0 25 50 75 100
Drainage Limits
National Forests
Yellow-stone National Park
Gros Ventre Range
Wind River Mountains
Wyoming Range
Green R.
Big Sandy R.
Bear R.
Divide
Green River
Cheyenne
Evanston
Great Salt Lake
Salt Lake City
Provo
Brown Park Dam Site
Vernal
Duchesne R.
Yampa R.
Lily Lake
Continental Divide
Kremmling Dam Site
White R.
White River Plateau
Water-shed 44,000 Sq.Mi.
Wasatch Mts.
Price R.
San Rafael R.
Roan or Book Plateau
Grand R.
Green River
Water-shed 26,000 Sq. Mi.
Cripple Creek
Gunnison R.
Uncompahgre Plateau
Aquarius Plateau
Escalante R.
Fremont R.
San Juan Mts.
Durango
Water-shed 24,500 Sq.Mi.
Water-shed 10,000 Sq.Mi.
Virgin R.
Paria R.
Colorado R.
San Juan R.
Rio de Chelly
Turley Dam Site
Blanco Canyon
Largo Canyon
Las Vegas
Grand Canyon
Kaibab Plateau
Marble Canyon
Santa Fe
Water-shed 25,000 Sq.Mi.
Little Colorado R.
Bull Head Dam Site
Mohave City
Juniper Mts.
San Francisco Mts.
Black Mesa
Rio Puerco of the West
Zuni R.
Holbrook
Woodruff
Water-shed 5,500 Sq.Mi.
Prescott
St. Johns
Mogollon Mesa
Datil Range
Williams Dam Site
Head-gate Rock
Parker
Harquahala Mts.
Black Point
Salton Sea
Laguna Dam
Picacho
Yuma Dam Site
Imperial
Calexico
Cut Holes
Gila R.
Phoenix
Salt R.
Roosevelt
Globe
San Carlos
Riverside
Florence
The Butte
Guthrie Dam Site
Solomonsville
Drainage Area above Yuma 262,500 Sq.Mi.
Water-shed 64,500 sq.mi.
Santa Cruz R.
San Pedro R.
Lordsburg
Continental Divide
El Paso
Colorado River
Total Drainage Area 265,000 Sq.Mi.
Nogales
GULF OF CALIFORNIA
NEVADA
UTAH
COLORADO
WYOMING
NEW MEXICO
ARIZONA
CALIFORNIA
MEXICO
PACIFIC OCEAN

FIG. 1.

Quite a little land along the San Juan, Animas, Pine, Florida, and La Plata Rivers, and the small tributaries of the San Juan, in Colorado, is now under cultivation, and also several thousand acres of valley land in New Mexico, but, as yet, irrigation has largely been confined to the bottom lands. The greatest probability of future irrigation development in this basin is in San Juan County, New Mexico, where it is said that probably 1 000 000 acres of fertile lands are excellently adapted for irrigation and for which the water supply is ample. The average annual run-off at Turley dam site is probably more than 1 000 000 acre-ft., the reservoir at that point having a capacity of 1 500 000 acre-ft., and the total stream flow at Shiprock, below the mouths of the Las Animas and La Plata Rivers, is more than 3 000 000 acre-ft.

In the Little Colorado River Basin there are scattered a few relatively unimportant patches of irrigated land, and the U. S. Reclamation Service has investigated and found feasible the irrigation of approximately 70 000 acres in the vicinity of Holbrook, by constructing storage reservoirs at St. John's and Woodruff, Ariz.

There are also irrigation possibilities in the Virgin River and Bill Williams Fork Basins, but their total area is relatively unimportant, as far as concerns their effect on floods, or the irrigation of lower lands.

There are excellent opportunities for irrigation in the Gila River Basin, chief of which are the projects examined by the U. S. Reclamation Service in the vicinity of Alma and Lordsburgh, N. Mex. At the latter point there are 250 000 acres of almost unbroken and very fertile land which could be irrigated by the stored water of the Gila River, although at considerable expense. Other good storage sites exist at San Carlos on the Gila, and at Roosevelt on the Salt, the latter having already been utilized by the U. S. Reclamation Service by building the famous Roosevelt Dam, behind which can be stored 1 100 000 acre-ft. of water. With this water, about 200 000 acres of land will be irrigated directly, and power will be generated for pumping water to nearly 60 000 acres more. In addition, there is an excellent reservoir site on the Verde River above McDowell, and large tracts of land on the Gila River in the vicinity of Solomonville and of Florence, Ariz., are now irrigated.

Along the Colorado River itself there are storage sites at Bullhead Point and at another point about 6 miles above the Laguna Dam

ear Yuma, while there are irrigable lands between Mohave and Yuma ggregating some 400 000 acres.

Table 1 is a summary of the areas above Yuma which are now rigated, in a technical sense, although much of this territory, no oubt, is watered in a very unsatisfactory manner.

TABLE 1.

District.	Acres.	Acres.
olorado River direct	19 000	
reen River and tributaries	255 000	
rand River and tributaries	305 000	
remont River	16 000	
n Juan River and tributaries	57 000	
ttle Colorado River and tributaries	12 000	
rgin River	16 000	
la River and tributaries	230 000	
attering (other tributaries)	7 500	917 500
Additional Irrigable Lands above the Yuma Valley.		
ove the Grand Cañon	450 000	
olorado River Valley below Mohave	400 000	
e Gila Drainage Basin	400 000	1 250 000
Irrigable Lands in the Delta.		
uma Project	90 000	
perial Valley in the United States	600 000	
perial Valley in Mexico	300 000	
her lands in Mexico—east of the Colorado	200 000	1 190 000
Grand total		3 357 500 acres.

TABLE 2.—Approximate Storage Possibilities of the Basin.

	Acre-feet.
reen River, including the Brown Park Reservoir site	3 000 000
and River, including the Kremmling Reservoir site	3 000 000
ttle Colorado	50 000
l Williams Fork	100 000
n Juan	1 500 000
rgin River	.....
la River	2 500 000
olorado, below Mohave and above Yuma	.....
Total	10 150 000 +

It must be borne in mind that all the figures in Tables 1 and 2 e for developments which are theoretically possible, and they would

have to be more or less seriously reduced to be correct for commercially feasible developments, on account of the excessive cost and the formidable character of the silt problem.

Discharge at Yuma.—Observations of the gauge heights of the Colorado River have been made by the Southern Pacific Company on its bridge at Yuma since 1878. The U. S. Geological Survey has maintained a gauging station at this point since 1895, using rating curves for discharge reductions until 1902, since which time careful current-meter observations have been made every 3 or 4 days. Table 3 contains the data thus collected for the 18-year period, 1894 to 1911, reduced to averages.

TABLE 3.—ANNUAL DISCHARGE OF COLORADO RIVER, AT YUMA, ARIZONA, FROM 1894 TO 1911, INCLUSIVE.

Year.	Mean, in cubic feet per second.	Total acre feet.
1894	7 400	5 390 000
1895	9 900	7 162 000
1896	9 000	6 515 000
1897	12 400	9 039 000
1898	9 100	6 581 000
1899	12 200	8 870 000
1900	9 400	6 798 000
1901	11 700	8 495 000
1902	8 400	6 127 000
1903	15 600	11 329 000
1904	13 900	10 119 000
1905	27 300	19 710 000
1906	26 800	19 475 000
1907	35 100	25 500 000
1908	18 900	13 700 000
1909	35 800	26 000 000
1910	19 700	14 335 000
1911	24 600	17 839 000
Mean	17 070	12 388 000

The minimum annual discharge was observed in 1894, and the maximum in 1909. The discharge has been strikingly greater since 1902 than for previous years, but too much dependence should not be placed on the data obtained prior to 1902, at which time very frequent current-meter observations were commenced. The lowest discharge was probably 2 400 sec-ft. in January, 1894, the average for that month being only 2 510 sec-ft.; the greatest was 149 500 sec-ft. on June 24th, 1909. The smallest total discharge for one month was 154 100 acre-ft. in January, 1894, and the greatest was 6 250 000 acre-ft. in June, 1909.

TABLE 4.—Mean Monthly Discharge of Colorado River, 1894 to 1911, Inclusive.

Month.	Cubic feet per second.	Total, mean monthly, in acre-feet.
January	7 340	450 400
February	8 370	466 900
March	12 830	787 800
April	16 380	973 200
May	34 280	2 104 200
June	50 500	3 000 000
July	29 630	1 819 200
August	13 560	832 700
September	9 880	586 900
October	8 460	519 000
November	6 660	395 900
December	7 060	433 200
Totals	17 080	12 369 400

The record for 1908 is given by months in Table 5 as typical of the monthly variation. The lesser disturbances caused by the floods from the Gila in the autumn are very well shown; in this case, the maximum discharge from this source occurs in December, instead of from the Colorado in June.

TABLE 5.—Monthly Discharge of Colorado River at Yuma, Arizona, for 1908.
(Drainage area, 260 000 sq. miles.)

Month.	Discharge, in Second-Feet.				Run-Off.	
	Maximum.	Minimum.	Mean.	Per square mile.	Depth, in inches, on drainage area.	Total, in acre-feet.
January	7 400	5 600	6 320	0.028	0.03	389 000
February	45 000	6 300	14 200	0.063	0.07	817 000
March	33 000	10 100	16 100	0.072	0.08	990 000
April	35 000	12 900	17 800	0.079	0.09	1 060 000
May	33 000	23 000	27 200	0.121	0.14	1 670 000
June	61 700	30 000	42 900	0.191	0.21	2 550 000
July	53 800	18 900	32 600	0.145	0.17	2 000 000
August	36 100	18 600	24 300	0.107	0.12	1 490 000
September	19 300	7 000	11 400	0.051	0.06	678 000
October	20 600	6 600	9 510	0.042	0.05	585 000
November	10 200	6 000	8 090	0.036	0.04	481 000
December	72 500	6 000	15 900	0.071	0.08	978 000
The year	72 500	5 600	18 900	0.084	1.14	13 700 000

Necessity for Storage.—The figures for the discharge at Yuma show that, in an ordinary dry year, the Colorado, without regulation, will serve an area not greatly exceeding 500 000 acres. On the other hand, in

an ordinary dry year, with fairly complete regulation—that is, with 2 000 000 acre-ft. of water storage—this river will serve 1 500 000 acres, and any supply held over from wet to dry years would add to the reserve. It is conservative to assume at present that no reservoir site on the Colorado below the Grand Cañon can be utilized, on account of the apparent absence of rock foundations for dams in the river, and, even if other things were favorable, the tremendous quantity of silt in the water means a heavy reduction in the reservoir capacity which could be obtained. Indeed, it has been seriously suggested that by the construction of a series of such dams, the silting up would in time create large areas of excellent land, one above the other.

Above the Grand Cañon, the Kremmling Reservoir site, on the Grand River, and the Brown Park Reservoir site, on the Green River, would together store approximately 4 500 000 acre-ft., and thereby add much more than 1 000 000 acres to the irrigable lands of the Arid West. When it is considered that the present irrigated area of Southern California, exclusive of the Imperial Valley, is less than 300 000 acres, the potentiality of storage along the Colorado is startling.

Another very important feature of water storage along the river is the marked effect it would have in decreasing the difficulty of controlling the Lower Colorado River. Levee construction and bank protection must obviously be designed to guard against maximum floods, and it is these which the storage basins would affect to the greatest degree. The completion of the Roosevelt Dam, which will hold back 1 100 000 acre-ft. on the Salt River, will in future undoubtedly reduce to a considerable extent the dreaded floods from the Gila River.

Rise of the Bed at Yuma.—If the measured discharge of the river at various heights is used in making a rating curve, and this curve is extended back, by means of the gauge readings, to 1878, the results would indicate that the quantity of water formerly passing Yuma was materially less than at present. As a matter of fact, the average low-water plane has constantly risen, and a comparison of the gauge heights by 10-year periods beginning with 1878 shows the following average elevations:

1878 to 1889	114.5 ft.
1890 " 1899	116.6 "
1900 " 1909	117.4 "

The low-water plane at the end of 1909, however, was 3½ ft. lower than during any of the six preceding years, which included the period of diversion into the Salton Sea. Indeed, it was lower, by more than 1½ ft., than 20 years ago, and only 0.8 ft. higher than during 1878-79. The reasons for this interesting condition of affairs will be considered later.

Following conventional practice, the endeavor was made for a long time to establish a rating curve for the Yuma gauging station, but this was found to be impossible. The reason is that the bed is eroded during high water and silted up during lower stages, thus fundamentally changing the cross-section, not only for different gauge heights, but for the same gauge heights at the beginning and end of

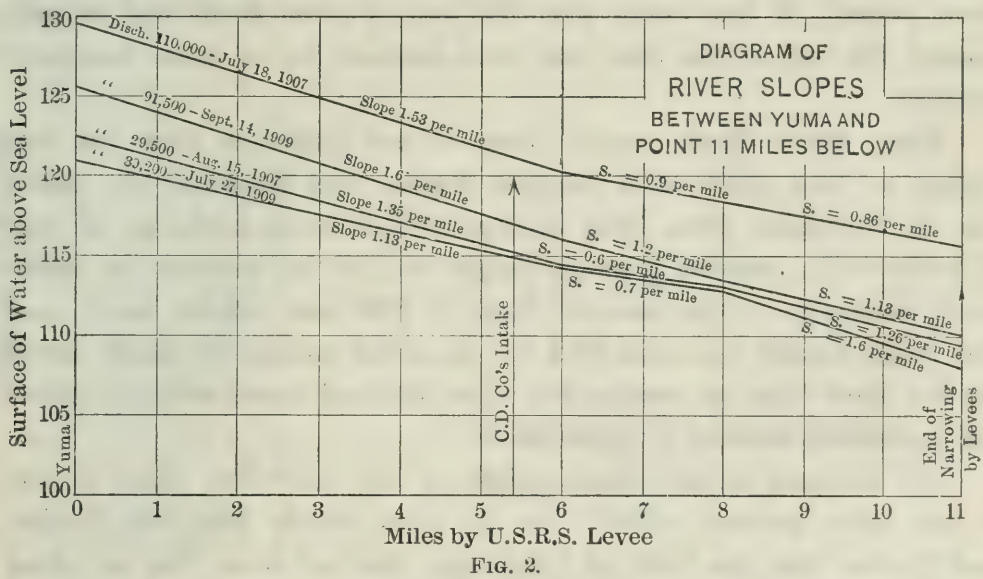

FIG. 2.

a high-water period. The reason for the exaggerated extent of such action is as follows: The Colorado at all times carries considerable silt, the quantity and character, of course, depending on the velocity of the water. Assuming a given discharge, and conditions of equilibrium, the bed of the river will have a given slope, the water will have a certain velocity, and will carry a certain quantity of sediment, none of which will exceed a definite size or specific gravity. If the volume of water increases, the water section and hydraulic radius will increase, and will result in greater velocity, which will give greater silt-carrying capacity. Conditions at the outfall or mouth are determined and temporarily unchangeable, therefore it follows that the grade of the river will automatically tend to flatten itself by

picking up additional quantities of silt and carrying them along. When the volume of water decreases, the velocity will slacken, resulting in carrying less silt, and the bottom will rise with increasing slope until equilibrium is again established. This condition of affairs results in surprisingly great changes at Yuma during long periods of high water. In 1907 and again in 1909 it was found that for an increase of 10 ft. in the gauge height there was a lowering of the bed of approximately 30 ft., making the total increase in depth of water almost 40 ft. In other words, the grade line drawn from the bottom of the channel at Yuma to the average water surface in the Gulf of California had 30 ft. more fall, from Yuma down, at the beginning of the summer floods of 1907 and 1909, than when the peaks had just been passed. A few weeks after the first of these floods had entirely passed, the bed of the river had been restored to its usual low-water position.

When flashy floods occur, there is not sufficient time for this action to take place to a marked degree, and therefore the flashy rise of November 28th, 1905, having an estimated discharge of only 115 000 sec-ft., reached a gauge height of 31.3 ft., whereas the maximum discharge in the summer flood of 1909 was 149 500 sec-ft. and the gauge height was only 29.2 ft. In other words, the flashy floods do not have time to render the river channel more efficient before the maximum demand is made on it.

The increase in the gauge height of the low-water plane is due to the same general action. As the river builds the delta farther and farther into the Gulf of California, the bed must rise all along the line, of course, taking averages of considerable periods of time. According to Capt. J. H. Mellon, of Yuma, Ariz., who for a great many years navigated the Lower Colorado, the delta fan has extended out into the Gulf more than 6 miles in the past 40 years. Assuming the fall of the river in the lower reaches at 1.2 ft. per mile, the rise in the bed should average 1.2 ft. in $6\frac{2}{3}$ years, or approximately 0.2 ft. per year. These figures are about what the hydrographs seem to show, namely, 2 ft. per 10-year period.

Effect of 1909 Flood.—The fact has been mentioned that the low-water plane at the end of 1909 was only 0.8 ft. higher than during 1879, and this becomes much more striking when the general elevations for the entire period are shown by a curve. There were two

PLATE XLII.
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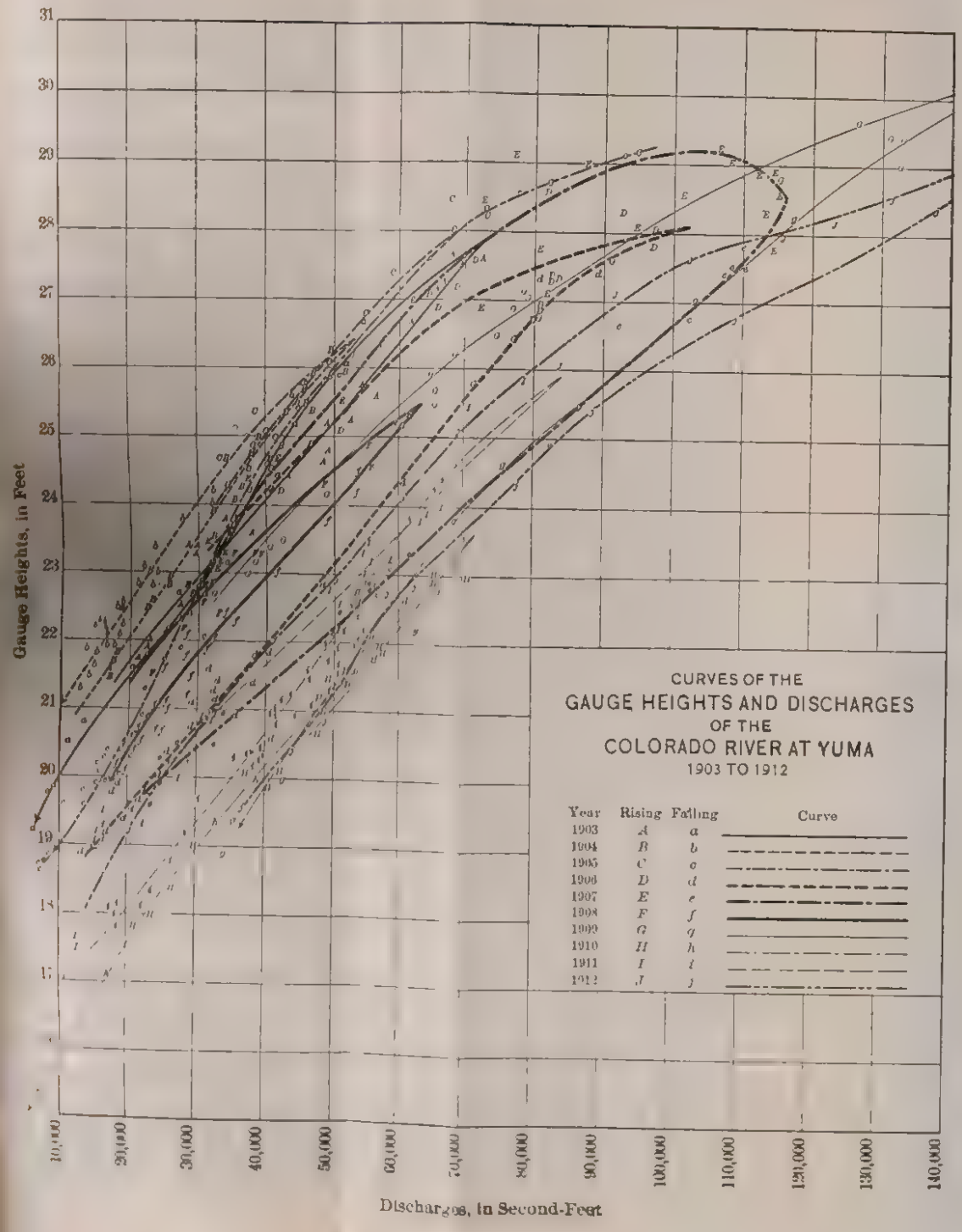
COLORADO RIVER DELTA.

12/4 A.



actors which tended to produce such a result: first, the diversion of he river through the Abejas to the west during the summer flood of 908, and the lowering of the river bed at that point; and second, the ffect of the Laguna Weir basin, which existed as such for the first me that year.

It seems very probable that the Abejas diversion was the smaller nfluence, in spite of the fact that at the time it was generally onsidered to be the only factor of importance. Undoubtedly, the ed of the river, and consequently the surface of the water, lowered apidly while the diversion was becoming an accomplished fact. The mount of such lowering could not have been more than a very ew feet at most, although it probably seemed much greater to ervous and frightened observers.

Doubtless it was an important factor that the Laguna Weir had een completed just before the beginning of that year's summer flood, nd created a reservoir having a capacity of perhaps 20 000 acre-ft. The waters of the Colorado, heavily laden with silt, were here stilled nd their contents deposited. The large volume of water which passed ver the dam—the greatest ever recorded on the river itself—contained ttle more silt than it would ordinarily during low-water stages. Consequently, it picked up and carried along the silt to an unprecedented xtent. As the waters receded, the bed was built back to a very nuch less extent, because there was still an extraordinarily small uantity of silt in the water. Indeed, during this one season, the asin formed by the Laguna Weir was completely filled and some 0 000 acre-ft. of mud were deposited out of the Colorado at this oint instead of being spread along the river bed thence to the Gulf.

Unfortunately, no sediment observations were made at Yuma dur- ng this flood period. Had this been done, the influence of the aguna Basin on the low-water plane would doubtless have been pproximately ascertainable. In any event, the gauge heights at uma, for discharges of 30 000 and 10 000 sec-ft., respectively, platted s ordinates, with the times as abscissas, as in Fig. 3,* for the period f 1902 to 1912, show very clearly that there has been no serious rade recession at Yuma due to the Abejas diversion.

* This method of platting seems to be the only one possible to show much relation, if ny at all, between gauge height, discharge, and time at the Yuma gauging station.

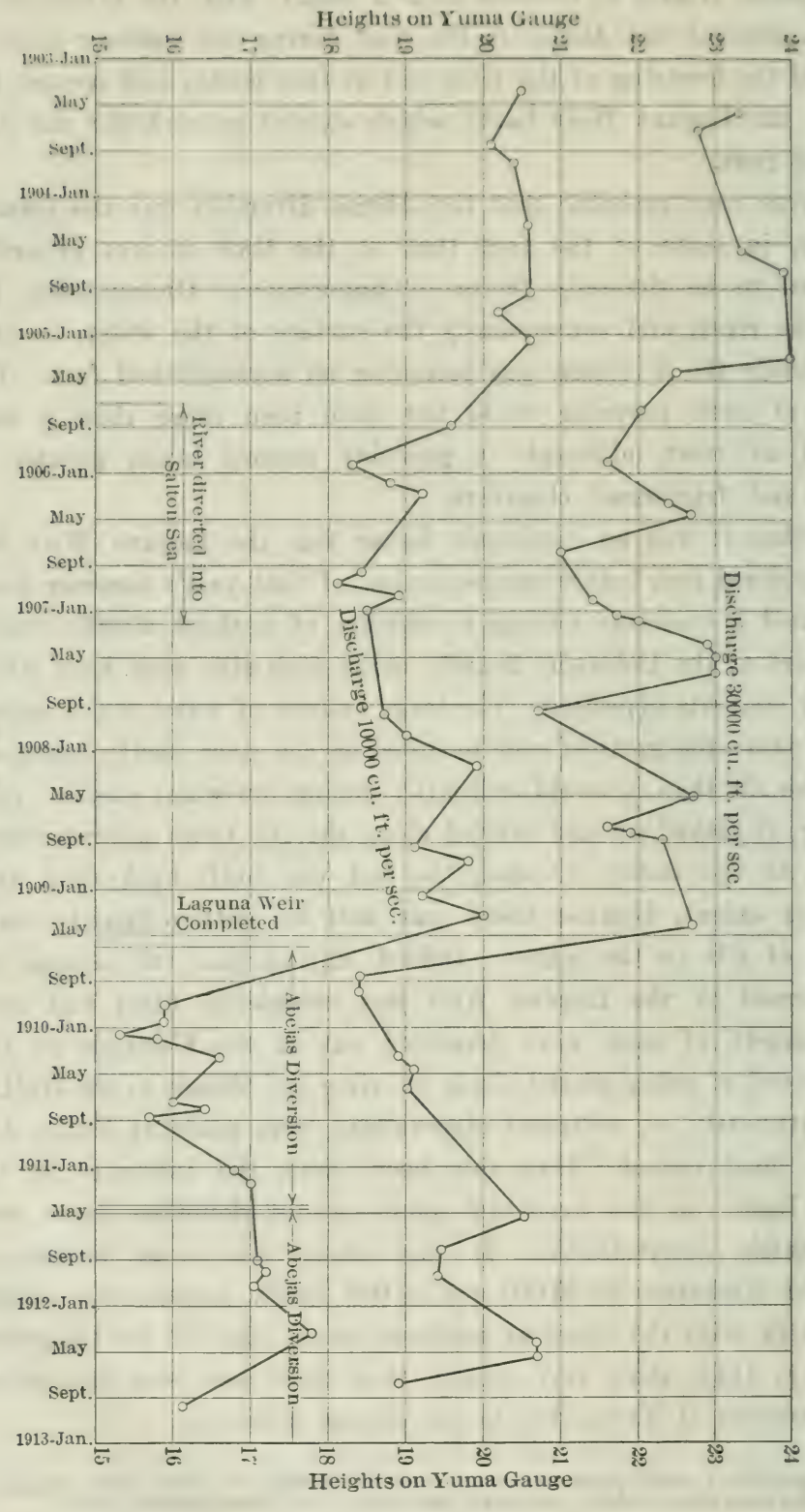


Fig. 3.

GAUGE HEIGHTS OF COLORADO RIVER AT YUMA
FOR DISCHARGES OF 30000 AND 10000 CU. FT. PER SEC.
JANUARY, 1903, TO JUNE, 1912.

Silt.—Professor C. B. Collingwood, of the University of Arizona Agricultural Experiment Station, examined the silt contents of the Colorado River water at Yuma for a period of 7 months, beginning with August, 1892. One pint of water was taken each day and evaporated, and the sum of the daily residues for each month was then weighed and analyzed. The results varied from a minimum of 1 part of sediment to 613 parts of water in January, 1893, to a maximum of 1 to 97 in October, 1902, and an average of 1 to 388, the ratio of dry material to weight of water being 1 to 277. The corresponding ratio in the Mississippi is 1 to 1 500; the Nile, 1 to 900; the Danube, 1 to 3 060. The average value of the fertilizing material was computed at $3.22 per acre-ft.*

Later, January 1st to December 31st, 1904, Professor R. H. Forbes, now Director of the same Experiment Station, made a careful study of the quantity of silt,† and the relation of irrigating sediments to field crops.‡ It was found that the quantity of silt varied from 84 to 3 263 parts per 100 000 by weight, and from 250 to 9 800 parts per 100 000 by volume, or roughly $\frac{1}{1\,200}$ to $\frac{1}{30}$ part by weight, and that acre-ft. of river water contained from a minimum of 1.14 tons to a maximum of 44.42 tons, and an average of 9.62 tons, of silt. Obviously, the total quantity of sedimental material cannot be obtained by multiplying the average sedimental contents by the total annual discharge, but the investigations were carried out in such detail that it was possible to compute the quantity of solid material from the discharge at the time, and in this way it was found that the total solid material carried past Yuma that year was 120 961 000 tons. The total discharge of the river during that year was 10 119 000 acre-ft., and the annual average for 1894 to 1911, inclusive, was 12 388 000 acre-ft. It would seem conservative to estimate that the average quantity of material would be as much larger than that delivered in 1904 as the discharge, on which basis the result would be $120\,961\,000 \div 10\,119\,000 \times 12\,388\,000 = 148\,084\,000$ tons. The specific gravity of the Colorado sediment is 2.65 and the weight of dry soil is 93 lb. per cu. ft., so that this quantity of material would make approximately 71 800 acre-ft. or 112 sq. mile-ft. of equivalent dry alluvial soil.

* These results are given in Bulletin No. 6 of the Arizona Agricultural Experiment Station.
† Bulletin No. 44.
‡ Bulletin No. 53.

Navigability.—In a technical sense, the Colorado River is navigable from its mouth up to Laguna Weir, and again from there to The Needles. This navigability was recognized when Mexico and the United States entered into the treaty of 1848 regarding the International Boundary Line. By the provisions of this treaty, neither country was to permit works which would interfere with navigation throughout that part of the river which is a common boundary. In a subsequent treaty (1853) this provision was abrogated, but the United States guaranteed in lieu thereof a free and uninterrupted passage of vessels and citizens as far as the river forms a common boundary. As a matter of fact, the swift, shoal waters and the shallow depth over bars in the river itself, together with a tidal bore at the mouth, where the range of tide exceeds 30 ft., has resulted in practically no commerce on the river below Yuma since the Southern Pacific Railroad completed its track in 1876. At various times the U. S. Army engineers have investigated the situation, but have always reported that the navigation interests were not sufficient to justify any expenditure for river improvement.

An Act approved April 21st, 1904, authorized the Secretary of the Interior to divert water from the Lower Colorado River for irrigation purposes and to construct a diverting weir across the river at The Potholes, or Laguna, in which no provision whatever is made for navigation.

Delta of the Colorado.

The Delta of the Colorado River of the West, at the head of the Gulf of California, lies approximately between the parallels of 32° and 33° N. and the meridians 114° 30′ and 115° W. It is partly north of the International Boundary Line between the United States and Mexico, and in larger part south of that line. Its area, including the Pattie Basin and the Cocopah Mountains, is approximately 6 000 sq. miles. It extends practically from the mouth of the Gila River, at Yuma, westward to the rocky walls of the San Jacinto Mountains and south to tide water of the Gulf, while on the north it blends with the depressed area below the sea-level, from which the ocean has been cut off by the deposits of the stream. Its general deltoid form is shown on Fig. 4, together with the course of the main stream and principal branches, sloughs, and overflow channels.

MAP OF
COLORADO DELTA
AND
IMPERIAL VALLEY
MILES
0 5 10 15
116°
115° 30′
115°
114° 30′
33° 30′
33°
32° 30′
32°
Indio
Coachella
Thermal
Mecca
Coachella Valley
Salton
Dos Palmas
Chuckawalla Mountains
RIVERSIDE COUNTY
IMPERIAL COUNTY
Durmid
Bertram
Frink
Volcano
Imperial Jc.
Flowing Well
Iris
Mammoth
Acolita
Mesquite
Glamis
Ruthven
Drylyn
Cactus
Ogilby
Pilot Knob
Ehrenberg
Colorado River
Picacho
YUMA CO.
IMPERIAL CO.
CALIFORNIA
ARIZONA
Laguna Dam
Potholes
Yuma Main Canal
Bard
Sellew
Powell
Andrade
Algodones
Yuma
Gila R.
Ivalon
Araby
Yuma Valley
86 Sq. Miles
Somerton
YUMA MESA
SALTON SEA
Agua Dulce
Soda Spring
Salt Spring
Soda Spr
McCain Spring
Seventeen Palms
SAN DIEGO CO.
IMPERIAL CO.
Mesquite Drill Hole
Hanna Drill Hole
Fish Creek Mt.
Superstition Mt. 764
Barrett Drill Hole
Coyote Mt.
Dixieland
S. D. & A. RY.
MESA
Coyote Wells
Yuha Drill Hole
Yuha Spring
UNITED STATES
MEXICO
Signal Mt. 2262
Pinto Mt.
Estelle
Bernice
Rockwood
L.W. Co. No. 8
Brawley
Keystone
IMPERIAL VALLEY
Imperial
Meloland
Holtville
Seeley
El Centro
Silsbee
Heber
Calexico
MESA
CALIFORNIA
BAJA CALIFORNIA
HIGH SAND MESA
Seven Wells
Paredones
Dieguenos
Cuervo
Packard
Pasqualito
Sesbania
Alamo R.
Imperial Canal
Cocopah
Hechicera
Bataques
Tecalote
Quail R.
Campo Lino
Volcano L.
Cerro Prieto
Borego Peak 3477
Hot Springs
Red Tank
Laguna Salada or Maquata
Cocopah Mountains
Womens Sprs
Pattie Basin
Sand Hills
Palomar Canyon
Abejas R.
Pescadero R.
Head of Tide Water
Col. R. Land & Irrigation Co. S.A. 686.000 Acres
Dry Bed
Colorado R.
Sonora Val. 312 Sq. Miles
Santa Clara Slough
DESTROYED
PARTLY DESTROYED
ARIZONA
SONORA
Cerro Mayor 3130
Laguna
Hardys Colorado
Arroyo del Bote
Route of Colorado R. 1910-12
W.S. El. 12.2 July 9-1911
Babcock
Tinajas Altas
Antelope Slough
Range Hill
MONTAGUE I.
GORE IS.
Gulf of California

FIG. 4

The Lower Colorado River.—The Lower Colorado River may be considered as that portion lying below the last narrows, at what is known as The Potholes—the location of the Laguna Weir, of the United States Reclamation Service. At this point the river debouches

FIG. 5.

upon the plain, and the valley on each side is bounded by diverging mesas. About 13 miles below, and just above Yuma, the Gila River joins it. The present location of this portion of the river is shown on Fig. 5.

Alignment.—Below The Potholes there are two controlling points: one is a peculiar knob of indurated clay at Yuma through the center

f which the river channel has passed since the first advent of the whites; the other is the granite hill known as Pilot Knob. The small minence at Yuma covers about 40 acres, and reaches a height of not nore than 100 ft. above the general level of the delta plain. A similar hough much smaller knob lies on the east bank of the river, about 000 yd. below and just to the south of the Southern Pacific Railroad Company's line and bridge, and is occupied by the reservoir and ettling basins of that company. These peculiar topographical features ontrol the river, with respect to its location at Yuma, and at Andrade, at the International Boundary Line, 8 miles farther down.

Grade.—The course of the river is quite winding, like every flashy, ilt-bearing stream with a relatively steep grade. The elevation at The Potholes is approximately 140 ft. above sea level, and the distance by the river is about 100 miles to the head of tide-water and 114 miles to the mouth at the Gulf. Thus the general average fall is approximately 1.3 ft. per mile.

Remarkable Vegetation.—Attention must be called to the dense nd varied vegetation throughout the region subject to the river's verflow. Arrow weed grows in nearly impenetrable jungles; mesquite nd screw-bean trees occur in forests of varying density on older stablished soil, while freshly deposited mud flats and banks are almost mmediately covered with seedling willows which quickly grow into eavy timber. For instance, Professor Forbes counted on an area ft. square 1 500 willow sprouts up to 20 in. high, and in another older rowth 90 young willow trees 20 ft. high.* Cottonwoods occur, but re not abundant. Along the river banks and sloughs there are lense thickets of common wild cane, which the Mexicans call *carrizo,* vith a densely matted root stock which affords great resistance to rosion of the soil because the plant spreads both by means of these oot stocks and by sending long slender stems or runners across the nud flats to distances of 20 or even 30 ft., and these strike root at very point, thus rapidly establishing the plant on newly made ground. In marshy locations are found great fields of a plant with an immense dible bulb used by the Cocopah Indians as a food, locally known as *ule.* In addition there is the *sesbania,* or so-called wild hemp, which is imited strictly to ground subject to overflow. It comes up from seed

* "The Lower Courses of the Colorado," R. H. Forbes, in *The Great Southwest*, Yuma, Ariz., Vol. 1, Oct., 1906, p. 2.

annually after the subsidence of the summer floods, stands in dense thickets from 5 to 20 ft. high, and is often square miles in area. This plant is also of interest because of its industrial possibilities. In general, the vegetation of the delta is remarkable for the manner in which plants of a kind mass together in areas, almost to the exclusion of other species, and for the remarkable density and immense areas occurring in continuous bodies, strips, and patches, particularly of willow, arrow weed, wild hemp, and *carrizo*.

Line Changes.—The entire Colorado River Delta has been said to consist of alluvial silt. When the river is low the water wanders in a devious way, along a very wide shallow bed in many places, and is everywhere confined by banks seldom exceeding 10 or 12 ft. high. During high stages these banks are overflowed at many points, and in the case of severe floods such overflow is practically general. The banks are thus wet and softened, and, when the river falls, caving and side-cutting proceed wherever the current is thrown at an angle against the confining banks, and often with startling rapidity. At the same time, the overflow water, being very heavily charged with silt, is checked by the dense, matted growth, and at once deposits its heavier particles, the smaller sizes being dropped a little farther down stream, and so on. Thus the country is built up most rapidly at the banks, and the land slopes away from the river at a constantly decreasing rate. Indeed, the theoretical cross-section of the land surface away from the stream is a hyperbola. Of course, these slopes are not identical at any two points along the river, but instrumental data at present available show the general average fall to be about 1½ ft. in the first 100 ft.; 3 ft. in the first 300 ft., and from 5 to 8 ft. in the first 3 000 ft.

Although the coarser silt deposits are thus found immediately at the river bank, there are several reasons why this has little practical significance. The overflow water gathers in little channels which follow the line of greatest slope and in general approximately away from and down stream, the direction being the resultant of the general grade parallel to the river, and of the slope locally from the river's banks to the abeyment on either side. Such overflow channels build up their miniature beds and banks exactly like the main channel; they join to form overflow creeks, and these in turn form the overflow rivers.

As the level of the river rises higher and higher by such overbank leposition, it is obviously only a question of time until an unusual lood will produce sufficiently high velocities in some of these overflow hannels to cause a recession of their grades extending through the iver bank, thus diverting a portion of the water through the new oute. Ordinarily, as the flood recedes, such breaks are clogged with rift and sediment, but sometimes the clogging action is not rapid nough to counteract the opposing forces successfully, and in this way adical and extensive changes of the river's course throughout the elta occur. Usually, these changes are in the nature of cutting off ends and thus shortening the channel.

At first thought it would seem that a diversion to the west would e a very probable occurrence during any great flood, because, with he constant extension of the delta southward, the gradient in that irection has become less, and to the west, more, until the fall toward he Gulf is much less than half as great per mile as that along former ourses to the Salton Basin. As a matter of fact, however, though he overflow waters go down these channels with considerable rapidity, he cross-sections for many miles from the river are exceedingly in-fficient, due to the dense vegetation, drift in the water, and occasion-lly, no doubt, to beaver dams.

In addition to the foregoing, there is another factor of importance: The bed of the main stream for quite a distance on each side of the nternational Boundary Line is excessively eroded during flood periods nd filled up during lower water stages, as has been fully explained, o that, with a given flood discharge in the river, the water going ver bank constantly decreases in quantity, in depth, and in velocity, nd it is only the overbank flow which is important in connection with he overflow channels.

Character of Local Silt Deposits.—These various actions result in he formation of numerous little pockets throughout the inundated reas, in which water is left standing after the recession of each flood. Wherever this occurs the very finest of the silt settles out and, on ecoming dry, cracks in large, somewhat hexagonal, irregular cakes. f the deposit is very thin, these cakes curl up when thoroughly dried nd are broken up and carried away as dust by the wind; but when very hick the cracks are sometimes 6 in. and even more in width at the op and extend down 4 or 5 ft. Dust and vegetation accumulate in

such gaping cracks, and the next flood deposits another layer of sediment. Then, the heavier materials having settled first, the result is that the pockets are arched over, thus producing underground interstices which must be carefully guarded against in levee and other earth construction for holding back water.

The character of such deposits depends on the nature of the silt carried in the overflow water, and thus it happens that it is usually possible by examination to determine whether a deposit was made by a flood from the Colorado, or from the Gila proper, or from the Salt River.

The rate of such local deposition is sometimes startling. One instance the writer observed was due to the flood of 1905 which caused the diversion of the river into the Salton Sea. This filled in the ground on the left-hand side of the break about 3 000 ft. from the old river bank over quite an area to a depth of 6 ft., or almost to the roof of an Indian's *ramada*.

In this manner the Colorado River, from its exit from the Grand Cañon near The Needles, Cal., to about 3 miles south of Yuma, Ariz., has wandered about between its eastern and western abeyments, and, where these were any distance apart, has built up alluvial valley stretches which are practically level transversely.

The Principal Overflow Channels.—The overbank flow of the Colorado on the east side does not gather into channels of importance because the eastern abeyment is near by, except far down the stream, where there is what is known as the Santa Clara Slough. This, doubtless, at one time was the river's main channel, and during the severe summer flood of 1907 it carried so large a volume of water that for a time it seemed probable it would again become the main outlet of the river to the Gulf. This slough is about 40 miles long, and empties into the Gulf about 20 miles southeastward from the present mouth. This and the other smaller high-water channels on the east side are of no material importance in the engineering operations along the river.

By far the greater portion of the delta cone lies on the west side, where there are five inundation channels of considerable importance. These are shown on Fig. 4. In their order down stream from Andrade, they are known as the Alamo, New, Paredones, Abejas, and Pescadero Rivers. Without a thorough knowledge of them and their relation-

FIG. 6.—TYPICAL SURFACE OF CRACKED ADOBE SOIL.

G. 7.—TYPICAL SUBSURFACE OF CRACKED ABODE MATERIAL. THE BOTTOM OF THE ROD IS 5.3 FEET BELOW THE SURFACE.

hip to the Colorado flood-waters, no satisfactory understanding of he problems of the Lower Colorado, and the endeavors to handle them, s possible.

The Alamo River, which was formerly often called the Salton or Carter's River, has its gathering ground in the northerly edge of the delta cone immediately south of Andrade. It follows somewhat closely he southern end of the sand hills, at times in a well-defined channel nd again spreading out in broad swamp sections, known locally as

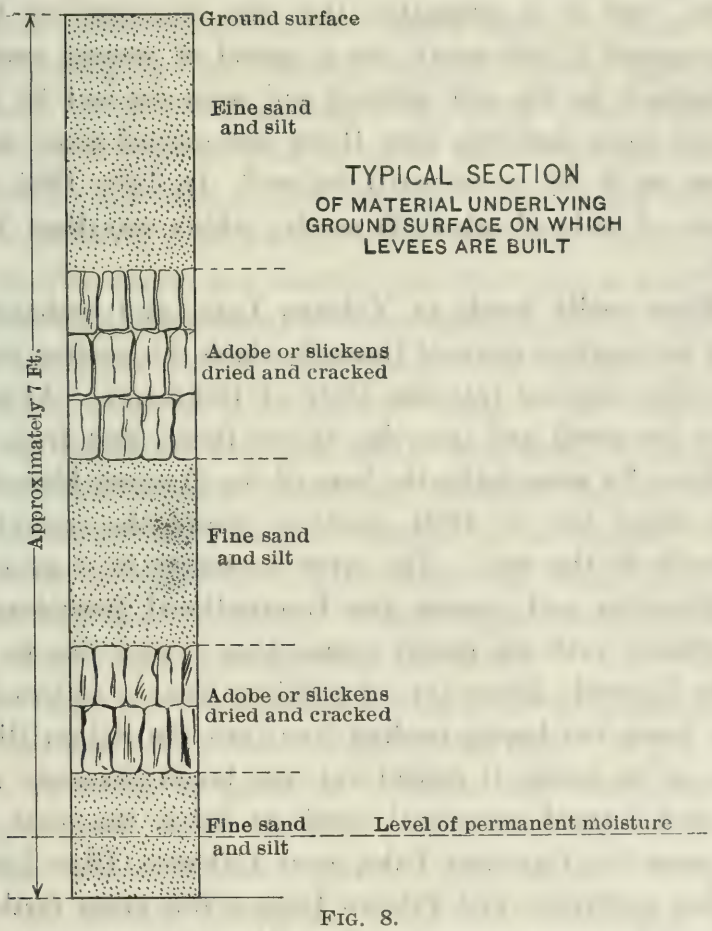


FIG. 8.

agunas. About 40 miles west of the Colorado it crosses the International Boundary Line, and occasionally its waters were doubtless carried clear into the Salton Sink. The swamp areas, *Las Lagunas,* were also drained in part by the Quail River, which emptied into the Paredones. Farther down the Alamo there is a low area to the south and west through which the overflow waters from the great flood of February, 1891, broke over from the Alamo into the New River, the main point being at what is known as Beltran's Slough. The latter

runs into the low region between the Paredones and the Alamo, and this drains into New River through the Garza Slough. It seems probable that in 1891, for the first time in many years, the overflow waters reached the New River channel directly *via* the Alamo, rather than *via* Volcano Lake. During this flood from the Gila and the later annual summer flood of the Colorado, sufficient water reached the Salton Sea *via* the Alamo and the New Rivers to cover approximately 100 000 acres in the bottom of Salton Sink to a depth of about 6 ft., and it is estimated that the discharge of both these rivers aggregated 17 000 sec-ft. for a period of several weeks. Well-defined channels in the soft alluvial soil were cut out by both these streams, and since then the New River has carried some water every flood season, as it did occasionally before.* In July, 1903, it reached a maximum of only about 4 000 sec-ft., which was then the largest since 1891.

New River really heads in Volcano Lake, and probably is what remains of an overflow channel through which the ancient inland lake, Lake Cahuilla, emptied into the Gulf of California. At present its grade is to the north and into the Salton Basin, and from the lake's edge it follows for some miles the base of the Cocopah Mountains until it reaches about the + 10-ft. contour, where the mountains turn rather sharply to the west. The river continues in a general northwesterly direction and crosses the International Boundary Line at Calexico, where, until the recent tremendous erosion due to the diversion of the Colorado River into the Salton Sea, it followed a gentle depression down the lowest median line into the Salton Sink. At a few places in its course it spread out into broad channels, a few feet in depth, and formed occasional ponds or lakes, the most important of which were the Cameron Lake, near Calexico, Blue Lake, a few miles farther northwest, and Pelican Lake, a few miles farther on. It is now a great barranca, averaging from 40 to 80 ft. in depth and 1 000 ft. in width, from a point about 6 miles southeast of Calexico to the Salton Sea.

The Paredones is the first drainage channel on the southerly slope of the delta cone, and within quite recent years had direct connection

* Old settlers in the vicinity agree in saying that in 1840, 1849, 1852, 1859, 1862, and 1867 large quantities of water reached the Salton Sea. In 1862 that body of water attained unusual size, and the flow in New River that summer was so great that it stopped the mail stage-line service between Yuma and San Diego for several weeks, and a flatboat was built to ferry across it, near Calexico.

with the Colorado River. This connection was automatically reduced to the very small channel which existed in 1906, when the levee construction then done fundamentally changed overflow conditions. The Paredones gathers the overflow water from a large area, and a few miles from the river becomes a channel of considerable width and depth, following thence along down an element of the delta cone. During the extraordinary conditions existing in 1905-06, it carried a very large quantity of drift, which, with the assistance of some beaver dams, accumulated about 7 miles above Volcano Lake, and resulted in enlarging the branches leading toward the south. The overflow water of this river gathers to the south in the Pescadero, and to the north joins with the similar water from the Alamo and runs in part to Volcano Lake and in part through Garza Slough to New River.

The Abejas River drains the overflow from the region immediately south of the Paredones, and empties into the western side of Volcano Lake. Since the summer flood of 1908, this channel has been carrying the entire low-water flow of the Colorado and the greater portion of the flood flow, which is the condition to-day. The reasons for this diversion and the efforts to stop it will be considered at length later.

The Pescadero, another important overflow channel, drains the region immediately below that unwatered by the Abejas. It empties into a network of channels which conduct the water from that part of the delta cone and including Volcano Lake, finally gathering into Hardy's Colorado and emptying into the Gulf.

Volcano Lake may be another remnant of the waterway through which the ancient Salton Sea drained to the Gulf. It is a flat basin, the bottom of which is about 22 ft. above sea level, and its high-water stage is about 35 ft. At such a stage it extends about 10 miles northwest and southeast, and is about 6 miles wide. It is fed by the Paredones and Abejas Rivers, the latter since 1908 being the course of the Colorado proper, and by the system of sloughs which form the Pescadero network and also serve as an outlet. It is on the summit of the low, flat divide between the Salton Basin on the north and the Gulf on the south, and thus its discharge is both toward the north and the south. From the size of the outlet channels it is obvious that the greatest discharge has in recent times been southward. Since 1908 a line of levees has prevented any water from passing into the New River and thence into Salton Sea; the lake's waters, therefore, go to

the Gulf through Hardy's Colorado, which is an important channel, averaging perhaps 500 ft. wide and 20 ft. deep at maximum stages, with a fall varying with the stage in the lake from less than 15 to more than 30 ft. in a distance of from 45 to 50 miles.

The engineering operations which resulted in the irrigation of the Imperial Valley and its threatened destruction by inundation at various times since, have in very large measure been concerned with the overflow channels just described.

Diversion to the West.—Regardless of the tendencies for and against a fundamental diversion toward the west, the Colorado continued to flow in its regular bed to the Gulf until 1905. There can be no doubt that the operations of the California Development Company, and particularly in making an artificial cut from the Colorado River into the Alamo Channel and the utilization of that channel as a main canal, rendered the diversion to the west at that point, when it broke through in 1905, very much easier and more probable of immediate occurrence. Nevertheless, the behavior of the river since that time indicates pretty clearly that a diversion to the west somewhere within the first 25 miles below Pilot Knob was just about due, under natural forces alone. The conditions of equilibrium had become unstable to a degree, and this is the condition in which they are to-day.

Mesa and Delta.—The high mesa land which forms the eastern abeyment below Yuma extends therefrom almost south and into Mexico. The river turns, crossing the valley almost from east to west for about 5 miles, until it reaches the foot-hills forming the west abeyment; then it turns more than a right angle, hugging these hills, to the International Line; and thence it flows for 80 miles, in a remarkably direct general line, but little west of south, to the Gulf. On the west side of the valley the foot-hills end at Pilot Knob, a small mountain at the International Boundary Line, and the low mesa begins. The edge of the latter runs southwest for 4 miles; then it turns sharply directly west for 25 miles; then again it turns sharply to a little west of north for 50 miles—the latter edge forming the east side of the cut-off portion of the Gulf, Lake Cahuilla.

It is thus in a sense almost proper to say that the Colorado Delta begins practically at the International Boundary Line between California and Lower California, and that, for the first 14 miles below that line, the river is running on the very edge of the divide of the delta

ones, on one side sloping northwest to the Salton Sea and on the other o the Gulf. Furthermore, from that point the river (until 1908) was n a ridge of its own making, which it was raising constantly, and vhich is quite close to the eastern abeyment.

Pilot Knob.—Pilot Knob is a small, detached, and relatively abrupt nountain lying just above the International Boundary Line on the vest bank of the river, and is one of the landmarks of the region. One f its rocky arms extends almost to the present west bank of the river. Fifty years ago the river had a pronounced bend, shown by the dotted ine on the map, Fig. 13, and hugged this rocky point until passing it. The time when the shift of the river took place is not definitely known, ut, very fortunately, at present the alignment here for several miles is lmost straight.

It is quite significant that Pilot Knob is the lowest point along the iver where a canal can be taken out for the diversion of water, with he diverting structure resting on solid rock. For this reason, it has een considered as a strategic factor in the irrigation of the Imperial Valley, but, in the writer's opinion, quite erroneously. The engineer- ng fetish of a solid rock foundation for structures for irrigation and ther purposes confining water, has resulted in needlessly spending mounts of money in the United States alone which must aggregate tremendous sum. Perhaps no case is more spectacular than that of Pilot Knob and its relation to the irrigation system of Imperial Valley.

Early Suggestions Regarding Salton Sea.—Almost the very first explorers were interested in the Salton Basin and its various possibili- ies. The ability to create an inland sea by diverting into it the water f the Colorado attracted much attention, and it was very seriously uggested because of a supposed advantageous effect that it might have n the climate of the entire region. On the other hand, the possi- ilities of irrigating the Colorado Desert by the waters of the Colorado, vhich has since been accomplished, were not overlooked, work having een done on many more or less serious propositions at various times.

Later Irrigation Projects.

In 1891 and 1892, the Colorado River Irrigation Company was formed. Mr. C. R. Rockwood was placed in charge of the engineering work, and, under his direction, the entire problem of irrigating the Colorado River Delta was carefully examined and the important

features fully worked out. The financial stringency of 1893 put an end to the operations of this corporation, and in 1894 Mr. Rockwood was forced to sue the company for his unpaid salary. In partial satisfaction of the judgment which he obtained, the engineering records and data were taken over by Mr. Rockwood, and the Colorado River Irrigation Company ceased to exist. Nevertheless, it is interesting to consider the plans then evolved by that corporation, or, more properly speaking, by its engineer, Mr. Rockwood.

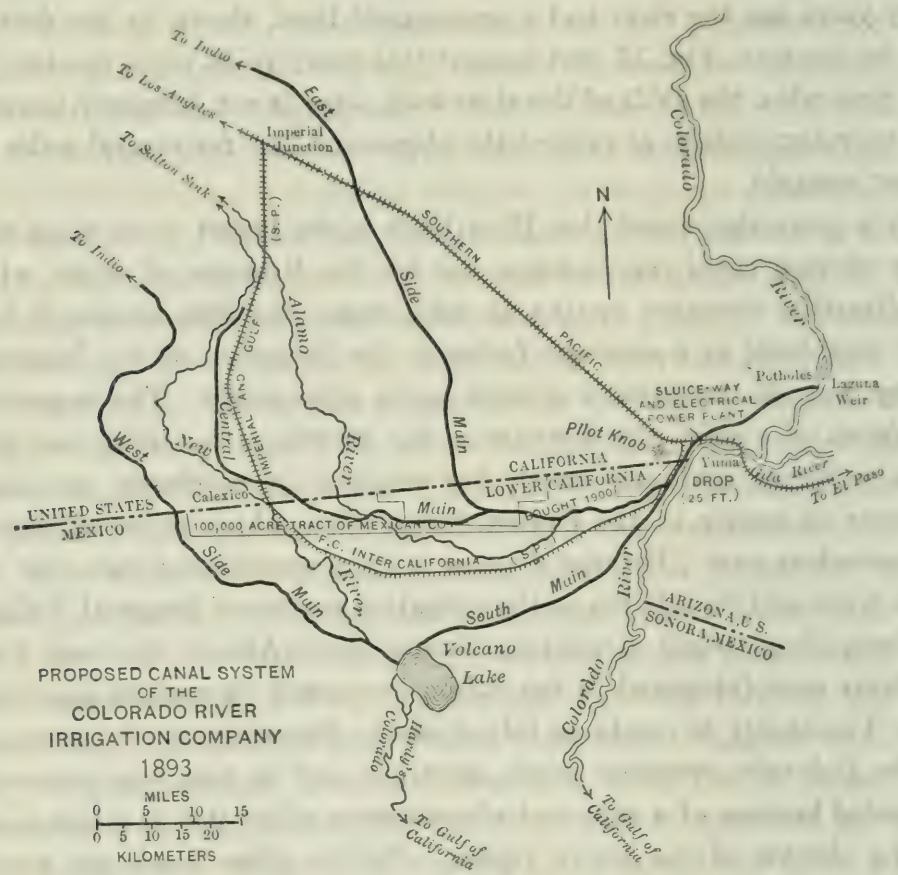


FIG. 9.

Plans of the Colorado River Irrigation Company.—These plans are outlined diagrammatically in Fig. 9, and show what is probably the ideal system of diversion and canals for watering all the land irrigable by gravity with the waters of the Lower Colorado. Events, however, shaped themselves so that the water for Imperial Valley has been, and is now being, diverted at Pilot Knob, and the U. S. Reclamation Service has built a diversion weir at The Potholes or Laguna to put water by gravity on all except the mesa lands in the so-called

Yuma Valley. Mr. Rockwood contemplated taking water out at The Potholes and installing in the main canal near Pilot Knob a sluiceway with which he intended to flush out the silt in the canal above which escaped being deposited in and removed by hydraulic dredges from a short enlarged section of the main canal immediately below the diversion point, where the velocity of the water would be reduced. The dredges were to be operated by electricity generated at the sluice-way. The maps showing the detailed surveys of these canals are now in the files of the California Development Company at its Calexico headquarters.

The California Development Company.—Mr. Rockwood, being thoroughly imbued with the practicability and advantages of the project to irrigate the Colorado Desert with the waters of the Colorado, undertook to carry it through, and finally did so by means of the California Development Company, under the engineering direction of, and with funds supplied by, Mr. George Chaffey, of Los Angeles, Cal. At the present time it is only important to say that, because of financial considerations, the engineering features were radically modified to diverting the water at Pilot Knob and utilizing a large part of the Alamo overflow channel as a main canal to carry the water around to the Imperial Valley, essentially as suggested by Lieut. Bergland in 1875-76. In this way the diversion work at The Potholes was eliminated, and a very cheap and quick method of getting water into the valley was arranged. By this decision the inclusion of the Yuma Valley as a part of the project was abandoned.

The Yuma Project, U. S. Reclamation Service.—As early as 1895 the Hydrographic Branch of the United States Geological Survey began stream gauging in California, starting with an allotment of $5 000. More recently, the California Legislature has aided in the work on the basis of appropriating sums equal to those set apart by the United States. At the present time daily discharge observations are made on about fifty typical streams. Hydrographic investigations throughout the Western States, not only helped to prepare the way for national irrigation, but resulted in acquiring such hydrographic data that when the Reclamation Act was passed, in 1902, the best opportunities for national irrigation projects were pretty generally outlined. On account of legal and social complications elsewhere throughout California, the Yuma Project was finally selected as the first to be

commenced by the Reclamation Service in that State. On April 8th, 1904, a board of seven engineers recommended this project; on April 21st, Congress authorized the Reclamation Service to take water from the Colorado and divert it by a weir which would close it to navigation permanently above Yuma. On May 10th, 1904, the Secretary of the Interior gave his approval, and an allotment of $3 000 000 was made.

There are approximately 75 000 acres of irrigable land under this project in Arizona, and 15 000 in California. Of this area, 98% is subject to the provisions of the Reclamation Act, the owners of private lands having signed the necessary agreements to limit their holdings to the size of the farm unit to be determined, and otherwise to conform to the regulations required by the Service. A Water Users' Association, consisting of the land owners of the project, handles the affairs of the district, from the farmers' point of view, and has contracted with the Secretary of the Interior to accept and use the water under the usual conditions fixed in such cases by the Government.

The Imperial Valley should logically have been included as a part of this project, particularly from an engineering point of view. However, water had been delivered into the Imperial Valley for almost 3 years when the Secretary of the Interior approved of the Yuma Project. In addition, there were complications—largely over-estimated and far more apparent than real—due to the fact that it is practically imperative to go through Mexican territory with canals to serve the American Imperial Valley. The project, therefore, was limited, for the present at least, to the irrigation of the Yuma Valley.

Fig. 5 is a map of the restricted Yuma Project. As it occupies a position on the river above that of the California Development Company's constructions, and for that reason in many ways has had a very important influence on the whole irrigation of the Colorado delta proper and related engineering problems, its essential features will be briefly described first. These are a diversion weir, and the levee, canal, and drainage systems. The diversion is by an overflow weir of the type developed by British engineers in their irrigation work in India, and improved and used later on the Nile. It is of loose rock, rests on a bed of river silt, is almost a mile long, very wide, quite low, and is in general an exceptionally interesting and expensive construction.

The next most unusual and interesting feature is the necessity for about 74 miles of levees to protect, from the overflow waters of the river, the greater part of the land to be irrigated. The canal system, with the exception of the siphon under the Colorado, is nothing out of the ordinary, and the same is true of the drainage system.

The Yuma Project, U. S. Reclamation Service.

The project has proved very much more expensive than was originally contemplated, the estimated cost being $3 000 000, whereas, the construction expenditures up to June 30th, 1910, were $3 617 472.71,* exclusive of maintenance and operation charges and $100 000 of the preliminary survey costs more properly chargeable to general investigations along the Colorado River than to the Yuma Project itself. Work on the project was reported as 80.8% complete, but this estimate was revised in April, 1911, and changed from 81.6 to 52.4%, making the proper percentage completed on June 30th, 1910, about 51.8. On this basis, the total cost will be $6 964 233, or approximately $77.25 per acre of irrigable land.

Laguna Weir.—The location and general design of this noted structure were determined by the character of the bluffs on each side of the last narrow point of the Colorado Valley, where they were almost a mile apart, and the fact that borings disclosed no bed-rock at reasonable depths in the river bed. Accordingly, it was decided to build a low, wide diversion weir of the so-called "Indian" type. The original design, as shown by Plate XLIII, was constructed with practically only one modification, namely, the interchange of the principal diversion from the Arizona to the California side.

Purpose.—The purpose of this structure, primarily, was to provide for silting out the heavier particles carried by the river, during flood periods especially, where such deposits could be sluiced out from time to time and in such a way that river floods would certainly carry them down stream.

Consideration will be given later to the silt problem, but it may be said that the only way of keeping the large, heavy, valueless particles of silt from getting into the distribution system, and clogging it, is to provide a settling basin where the water for a short time will either be practically still or the velocity reduced to not more than

* Ninth Annual Report, U. S. Reclamation Service, Washington, 1911, p. 81.

0.5 ft. per sec., with freedom from eddy currents. Such deposits may be removed either by sluicing out with large volumes of water at a high velocity, or by using pumps, dredges, or some other kind of machinery. It was estimated that in the main canal, originally designed to carry 1 600 sec-ft., the volume of wet silt to be removed therefrom daily was approximately 17 000 cu. yd. The sluice-way method of doing this means a higher initial cost and certainty of success, as compared with the very much lower cost, greater maintenance and operation charges, and somewhat less certainty of operation, for removal by machinery. The Laguna Dam (or rather weir) consists essentially of sluice-ways at each end of the structure, with a barrier between to hold up the water and afford a head for sluicing.

Sluice-ways.—The sluice-ways are controlled at their lower ends by large, vertical, steel-plate gates which are raised and lowered by electric machinery. The method of operation is to close the gates and cause the water in the sluice-ways above them to become practically still. The water thus held back quickly drops its heavier silt, while the canals are supplied through flash-board regulator gates in the outer sides of the sluice-ways, these gates being so long that a thin stream of water running over the tops suffices, and no water from near the bottom, where the sediment is greatest, is ever taken. From time to time, as may be necessary—and this varies greatly, depending on the stage of the river and the quantity of heavier silt particles carried—the gates are rapidly raised, and, with a fall of about 10 ft., the water rushes through the sluice-ways carrying away the silt deposits and dropping them a short distance below. During the annual and other floods, these deposits are taken up by the river and carried down stream. These sluice-ways are built through rock, their floor elevations being 13 ft. below the crest of the weir. They are lined and paved with concrete, and constitute a very massive and beautiful piece of work.

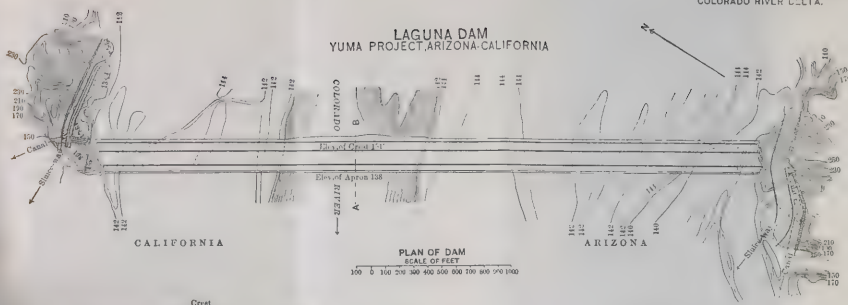
The Weir.—Between these sluice-ways the weir is built, the slope of the face being very flat, only 1 to 12, and capped with a concrete pavement, 18 in. thick, except a small portion which is paved with rough stones from 2 to 3 ft. thick. The crest was 10 ft. above the low-water mark at the dam when the structure was started,* and the top

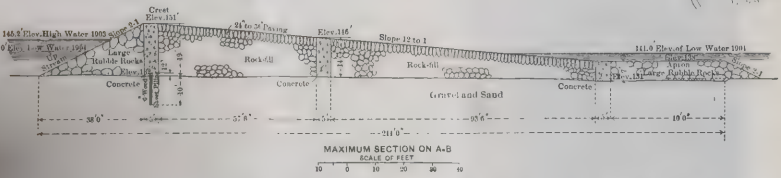
* See discussion of variation in elevation of river bed at different times and different seasons, and the consequent low-water mark.

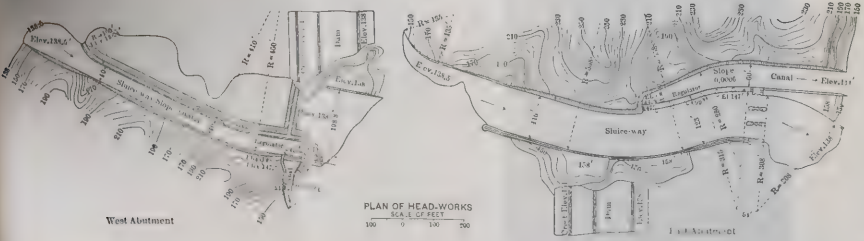
PLATE XLIII.
TRANS. AM. SOC. CIV. ENGRS
VOL. LXXVI, No. 1270.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.

1296A

LAGUNA DAM
YUMA PROJECT, ARIZONA-CALIFORNIA







of the down-stream wall is 3 ft. below; thus the total fall of the face is 13 ft.

The weir was constructed simultaneously from each end, and a gap of 800 ft. was left in the center of the river channel. The original plan for completing this gap was by building upper and lower coffer-dams with piling, brush, and sand bags, but this was changed and finally the barrier rock fill dam, developed by the operations of closing the first and second breaks described later, was utilized. Before the central section was filled in, the sluice-ways had been excavated and completed, the total capacity being more than enough to carry the low-water flow of the river. Rock was obtained in part from the excavation of the sluice-ways, and in part from the hills on each side; it was loaded on cars with derricks and steam shovels, and hauled by dinky locomotives to the various portions of the work. Coffer-dams made of quarry spoil were extended out into the stream, and inside these large pumps were used to clear of water. As much excavation as possible was done with teams and scrapers, and the remainder was taken out by suction dredges and pumps. Sheet-piling and the parallel concrete walls were then built, and the rock filling between was put in, followed up by the concrete surfacing. The actual quantities used exceeded the original estimates considerably; they are given in Table 6.

TABLE 6.

Rock excavation	444 640 cu. yd., or about	146% of estimates.
Earth excavation	346 930 " " " "	123% " "
Rock in dam	375 018 " " " "	123% " "
Concrete*	76 066 " " " "	280% " "
Sheet-piling	82 779 lin. ft. " "	156% " "
Rock pavement	Insignificant—decrease.	100% " "

* In place of rock paving, the concrete surface was substituted.

On March 15th, 1905, bids for the construction of the Laguna Weir were opened, but those submitted were rejected and the work was re-advertised. Proposals were again opened on June 5th, and on July 6th, 1905, the contract was awarded at the following prices:

Rock excavation	$1.30	per cu. yd.
Earth excavation	0.30	" " "
Rock in dam	0.35	" " "
Concrete	4.00	" " "
Sheet-piling	0.40	" lin. ft.
Laying pavement	1.00	" sq. yd.

The contract required the work to be finished within 2 years, which would mean just at the time of the peak of the summer flood of 1907. These prices, on the estimated quantities, made the bid amount to $797 650. There were seven other bidders, whose figures ranged up to $1 030 117.50. The Reclamation Service, under the specifications, supplied the cement to be used. On February 28th, 1906, the same firm was awarded the contract for furnishing and erecting the sluice-gates, regulator-gates, and operating machinery for the main sluice-ways and head-gates, the bid being $65 900. The contractors began work on July 19th, 1905, and a year later had completed 26.4% of the work.

As the quality of the rock obtained was much poorer than had been anticipated, the Board of Engineers of the Reclamation Service modified some requirements in the specifications and contract which resulted in increasing the contract price by $331 486, or to $1 129 136, and extended the specified time for completing the structure from July 19th, 1907, to January 19th, 1908. On January 23d, 1907, when about 34% had been completed, the work was taken over by the Reclamation Service direct. On July 1st, 1907, 52% of the work had been finished, a year later 77% was done; and it was practically completed in March, 1909, just before the summer flood of that year began.

The Reclamation Service gives the following costs* of the Laguna Weir and the sluice and regulator works:

Laguna Weir	$1 672 168.20
Sluice and regulator works	345 295.92

Other recent operations along the river have shown that a structure serving every purpose of the Laguna Weir could have been built by methods now well known at far less cost. The building of rock fill dams in the bed of such a stream as the Lower Colorado was considered impracticable until the work of re-diverting the river developed such method. However, it is now evident that it would have been far simpler, quicker, and cheaper to have developed rock quarries, thrown trestles across the bed of the stream, and dumped rock therefrom to form a wide rock fill dam, without any concrete walls whatever, and covered the top with concrete. There would be no difficulty in beginning the construction of such a dam in the center of the stream and causing the river itself to excavate its bed opposite the rock fill as the latter should be built forward. In this way the excavation

* Ninth Annual Report, United States Reclamation Service, Washington, 1911, p. 82.

would have been made to a little greater depth than the bottom of the concrete walls actually put in. The rock for such a purpose would by preference be graded, so that quarry spoil would in no way be objectionable, and rock material obtainable in the adjoining hills could be blasted out in large quantities, loaded with steam shovels, and consequently obtained and handled very cheaply. A structure having essentially the same top dimensions and surface covering, and extending deeper into the river bed than the existing one, would in this way have cost far less and be even more secure from failure. There would be practically no seepage through or under such a dam or weir, as the similar constructions, very much thinner and sustaining much greater heads, which closed the first and second breaks, seem to be absolutely water-tight.

To the cost figures should be added the proportional share of the total administrative and general expenses. Such administration figures are given as $179 021.43, to which should properly be added at least $75 000 of the item: "Preliminary surveys previous to selection of project—$174 735.83," or a total of $254 021.43. These are probably the approximate general expenses to be distributed over expenditures totaling $3 717 472.71, or 6.89 per cent. On this basis, there should be added to the cost, for general expenses, $114 243.24, or a total for the Laguna Weir proper of $1 786 411.44, and to the sluice and regulator works $23 583.71, making their total $368 879.63, or a total of $2 155 291.07, not including the loss of $400 000 said to have been sustained by the contractor, which would raise the total to $2 555 291.07.

The result is a magnificent and permanent head-works for taking water from the Colorado to irrigate by gravity about 75 000 acres of land in the Yuma Valley; and, at some future time, this structure may serve as well for diverting water to irrigate the entire Colorado Delta. Its very great cost, however, raises the question as to whether the silt problem could not have been solved in a more economical and equally satisfactory manner by pumping depositions thereof, in an enlarged section of the canal, back into the river, with suction dredges. This question cannot be determined until the maintenance costs of the sluice-ways and diversion weir are shown by experience, and the total costs and results of handling the silt with dredges, as is now being done at the California Development Company's head-works, have been ascertained for a considerable period.

Levee System of the Yuma Project.—Practically all the valley lands in the Yuma Project are subject to overflow, so that a general and comprehensive system of levee protection is necessary. Fig. 5 shows this system, practically all of which has been completed. In general, the designs were for dikes 4 000 ft. apart along the Colorado and 3 200 ft. apart along the Gila, with a height of from 4 to 5 ft. above the high-water marks; as constructed, however, there are long stretches along the Colorado where the levees are only from 1 600 to 1 800 ft. apart.

The first levee construction was in accordance with the usual Mississippi River practice. The ground was cleared, stumps and roots were grubbed out, the base was plowed, and the levee was built with earth taken from borrow-pits on the river side. These borrow-pits were about 400 ft. long in the direction of the levee, with cleared traverses between about 12 ft. wide; 40-ft. berms; allowable depths of pits, 2½ ft. at the side nearest the levee and 3½ ft. at the farther side; levee top width, 8 ft; side slopes, 3 to 1 on the river side and 2½ to 1 on the land side. No muck-ditching was done.

The first stretch of levee constructed was 10 miles long, extending south from Yuma along the eastern bank of the river. In this section the current along the levee face was generally as little as would be expected anywhere on the project. Nevertheless, experience soon showed the desirability of an elaborate system of brush abatis work, a sample of what was put in here being shown by Fig. 10. At many points where any considerable quantity of water had come against the face of the levee the borrow-pits had cut together, the traverses having quickly been cut through and the breach widened more or less seriously. As it was expected that the river would fill up these borrow-pits with silt in the first few floods, such a result was disappointing.

It seems that no trouble was caused by the absence of muck-ditch protection under the levee. This must have been due to the fact that the ground where the levee was located was uniformly favorable. In the fall of 1906, however, the levee system of the project was extended some miles southward along the river, and the flood which occurred on December 7th, 1906—which got under the newly constructed dikes on the west side of the river in many places and resulted in the second break or crevasse to the west—caused several breaks in this new section, due to the lack of muck-ditches in unfavorable ground.

FIG. 10.—TYPICAL ABATIS WORK ON LEVEES OF YUMA PROJECT, U. S. RECLAMATION SERVICE.

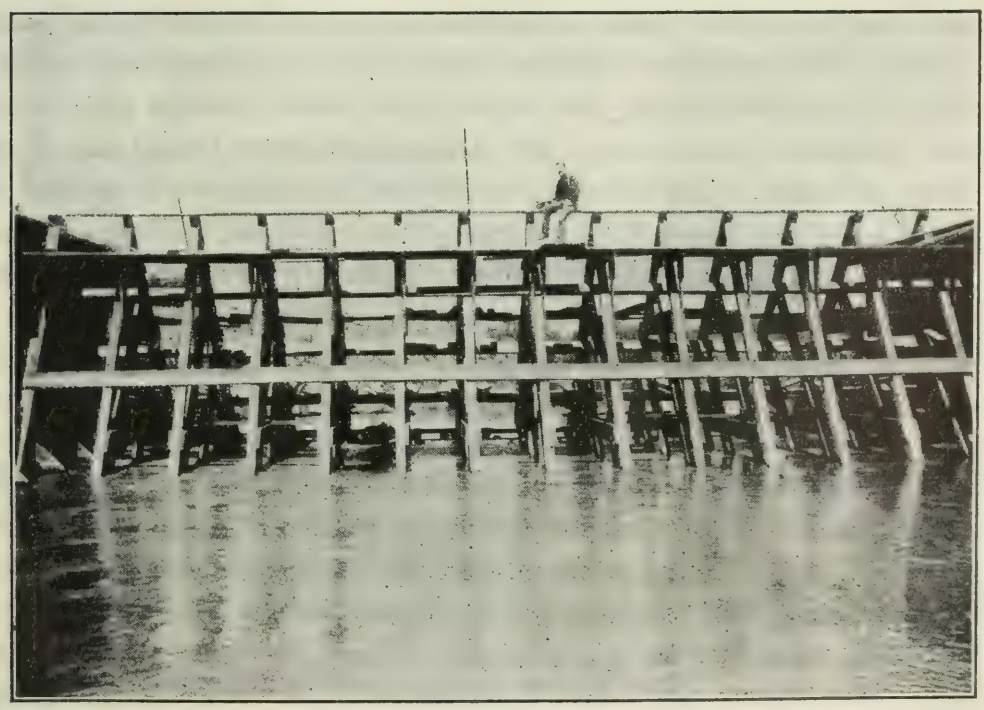


FIG. 11.—ORIGINAL INTAKE (CHAFFEY) GATE, IMPERIAL CANAL, COMPLETED IN 1901.

Experience with the levees, including the effect of this last-mentioned flood on the levee system of the project and on the levee work done on the other side of the river, caused a fundamental change in the design. In January, 1907, a Consulting Board of Engineers from the U. S. Reclamation Service was appointed to consider the matter of levee construction being done with money advanced by the Harriman interests on the west side of the river, and its recommendations are given later. Up to that date, 21 miles of levees had been constructed on the Yuma Project, extending from Yuma southward 15 miles and eastward along the south bank of the Gila 6 miles. All construction thereafter has been in accordance with the recommendations of this Consulting Board for the levees of the west side of the river, the essential features of which are "interrupted or checker-board borrow-pits" on the water side of the dikes, muck-ditches wherever test-pits show the necessity, and a large quantity of brush abatis work.

In 1907, a railroad track was laid in large part on the levee, from the Laguna Weir to Yuma on the California side of the river, chiefly for the purpose of hauling materials and supplies to and from the Laguna Weir. The Southern Pacific Company owns and operates this track as a branch line, thus serving an area which will be under intensive cultivation very soon, and greatly facilitating levee maintenance and repairs. Over this track a very large quantity of quarry spoil was hauled from the Laguna Weir construction work and used to blanket the river side of this levee to a point below where the swiftest water along its face is to be expected. None of the other levees of the project has any blanketing or any track on top.

Canal System of the Yuma Project.—Fig. 5 shows the general layout of the canal system of the Yuma Project as it is planned at present and in considerable measure constructed. The principal main canal is on the California side, and has a capacity of 1 700 sec-ft. The main canal on the Arizona side will irrigate only the land north of the Gila River. Water for irrigating the land lying east and south of the Colorado and below the Gila is to be carried under the river at Yuma in an inverted siphon, 1 000 ft. long, 14 ft. in diameter, about 50 ft. below the bed of that stream, and having an estimated capacity of 1 400 sec-ft. This siphon is now under construction. The original plan was to serve this territory with water taken from the

Arizona end of the dam and carried across the Gila River in four reinforced concrete tubes with a combined capacity of 1 300 sec-ft. and laid 3 ft. below the river bed. This crossing was abandoned because the difficulty of holding the Gila River banks at the ends of the underground siphon was considered too great. There is practically no danger of this kind in crossing the Colorado with the siphon, because of the little eminences of quite hard material which control the location of the river at this point.

The design of this siphon, the investigations of the material in which it is located, the first methods of construction used, the difficulties encountered, the changes in plans and methods, with the reasons therefor, the methods of doing the work finally adopted, and the time and cost figures, are all interesting in the extreme, but will not be given here for several reasons, chief of which is that, on the completion of the Yuma Project, it is hoped the work will be described at length in a paper by the Project Engineer, F. L. Sellew, M. Am. Soc. C. E., or some other engineer of the Reclamation Service. Only such general description is here given as seems desirable to make quite clear the effect of the project itself directly and indirectly on the irrigation of the delta.

The total acreage which will ultimately be irrigated by the Yuma Project is given in the reports of the Reclamation Service as 90 160. This includes 17 000 acres of mesa lands which lie too high to be reached by gravity from the principal canal system. It is intended to develop 1 000 h.p. at the drop in the main canal, and with this to operate pumps to raise water for the higher distribution systems.

At present the main canal from the Laguna Weir on the California side down to the California shaft of the river crossing at Yuma is under construction, and quite a little main and some lateral canals lying between these points and behind the line of the levees on the west side of the river have been completed and are in use. Such canal construction, and particularly the checks, head-gates, etc., are models of their kind, being of concrete and of the latest and most approved type. Up to the end of 1907, there were no canals on the California side of the river; indeed, practically all the area was contained in the Yuma Indian Reservation, which has since then in part been apportioned to individual Indians and in part, 6 500 acres, on March 1st, 1910, opened to entry and quickly taken up

by white settlers. Two weeks later water was turned into the reservation canals, and rapid progress is being made in developing the region. On the east side of the river below Yuma about 8 000 acres are being irrigated through small canal systems which have been in operation for a long time and were taken over by the Reclamation Service since the creation of the project. The total acreage of the project to which water could have been supplied was about 16 000 acres; about 10 000 acres were actually irrigated during the season of 1911.

Drainage System of the Yuma Project.—As has been said, a large acreage of the Yuma Project is subject to annual overflow, and lies behind the levees. The water-table throughout practically the entire region rises and falls with surprising rapidity during all floods which are long in passing. Thus it is that during May, June, July, and August particularly, the water-table rises so near the surface as to result in rather high alkalinity in the soil. The river water which will be applied for irrigation also carries a considerable, though not serious, quantity of soluble salts. Evaporation takes place from land surfaces very rapidly in such a hot country, and when water is on the surface, or approaches so near it that capillarity makes connection between the water-table and the surface of the ground, the quantity evaporated is excessive, and the salts contained are left behind, largely in the top layers. Therefore, efficient drainage is very important. It is made even more necessary because the rainfall is really inappreciable, having been less than 3 in. per annum for the past 15 years, and causes very little leaching and washing away of alkaline deposits. In passing, it is important to say that, very fortunately, the alkali of the valley lands is peculiarly a surface accumulation, often being confined to the very upper layers, usually to the first 2 ft. in depth, and seldom being found at depths exceeding 6 ft.

The Yuma Project, therefore, includes plans for an elaborate and efficient system of drainage canals which will be doing the maximum amount of work during the annual summer floods of the Colorado. It is planned, where necessary, to pump such drainage water over the levees and back into the river. This drainage system has not been constructed, and indeed the detailed plans may not yet have been worked out, but it is desired to state here that arrangements have been made for drainage in the Yuma Project, and results along that line must be obtained in the Imperial Valley sooner or later.

Imperial Valley Irrigation Project.

In 1893, Mr. Rockwood found himself in possession of much engineering and other information regarding the irrigation of the Colorado Desert with the water of the Colorado River, in lieu of salary for a considerable time as Chief Engineer of the Colorado River Irrigation Company, and had a firm conviction of the project's possibilities. For more than 7 years he endeavored to finance the work, both in the United States and abroad. Many people suggested the irrigation of the Colorado Desert, as already mentioned, but Mr. Rockwood and associates actually brought it about. His very interesting story of the enterprise,* unfortunately, is accessible to relatively few people. In spite of his later mistakes, Mr. Rockwood is certainly entitled to much credit and reward for his efforts, which, practically speaking, were finally crowned with complete success.

Engineering Features.—The engineering features of irrigating the Imperial Valley from the Colorado River can now be much better understood than was possible in 1900. The experience of 10 years is always of value, and was particularly so in this case. The fall of the ground was known, and to divert the water and conduct it to the broad, ideally lying tracts of land to the west of the sand hills was obviously practicable. There were, however, two especially serious problems: the danger of diverting water from a wide, erratic stream flowing through a shifting channel along the top of a ridge of loose alluvial silt; and the difficulty of keeping open canals which carried water so heavily charged with silt.

Diversion.—The impossibility of properly financing the enterprise absolutely forced the abandonment of the idea of diversion at The Potholes, with its opportunities for settling basins and sluice-ways to care for the silt *en route,* and made the diversion at the rocky point of Pilot Knob practically unavoidable. It was always the idea to have a head-gate founded on solid rock. At the last, it was found impossible to obtain the money, even for this construction, but the diversion point was located there, with the intention of utilizing this rocky point of Pilot Knob for head-works, in the very near future, when the financial status of the company might permit.

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* "Born of the Desert," by C. R. Rockwood, in the Second Annual Magazine Edition *Calexico Chronicle*, Calexico, Cal., May, 1909.

Flood Protection.—It does not seem to have been realized, at the time, or indeed by any one until the diversion into the Salton Sea was actually an accomplished fact in 1905, that there was any really appreciable danger to the Imperial Valley by flood-waters from the Colorado. The writer hopes especially, that the discussion will bring out any contradiction of this statement which may be successfully maintained. Of course, it was known that large quantities of water had been carried through the New and Alamo Rivers into the Salton Sea in 1891, and also by the New River earlier, especially in 1862, but the channels had not eroded to any marked degree at the gathering ground along the Colorado River bank, but, on the contrary, had automatically closed. Instrumental data regarding that portion of the delta cone which is subject to overflow were entirely lacking, and indeed, little other reliable information about the region was available. It was planned to build levees along the river side of the canal with the material taken from the latter, but the purpose of these levees was to protect the canal itself from danger, and not to keep the flood-waters which might enter this waterway from enlarging it to a dangerous degree. Of course, any risk of the river being diverted into the Salton Sink, and soon inundating the entire Imperial Valley, involves the same risk to the irrigation project as such. Otherwise, such risk should obviously not affect an irrigation company in any way, unless its operations and constructions have an appreciable effect on such river diversion.

Silt.—With this means of diversion it was necessary to let the silt-laden river water enter the canals directly, and depend on keeping them open by dredging, erosion, etc. The chief difficulty obviously must occur in the first stretches of the waterway, due to the rapid deposition of the heavier or sandy particles of silt which the river water carries during flood stages of excessively high currents, and which drops down almost at once when the velocity decreases to, say, 3½ ft. per sec. After such clarification, it is possible to design and operate canals in the Colorado Delta, as well as in India and elsewhere, so as to insure the carriage of the remaining finer silt into the smaller laterals and to the irrigated land. The first mile of the canal, therefore, was designed with a large cross-section so as to secure the deposition of this heavier silt there, where it could be removed by dredges.

Alkaline Lands.—From a farming point of view, a difficulty which was not given very serious consideration was the relatively high alkalinity in the upper layers of the soil throughout practically all the Imperial Valley. Wherever water came in contact with the ground, it was observed that vegetation at once sprang up like magic, and it was assumed, from this and from the obvious methods of its occurrence, that the soil must be exceedingly fertile and admirably adapted for general agricultural purposes. In one sense, a very serious mistake in this way was not made, for agriculture of almost unparalleled success has been followed for the past 10 years, with only at rare intervals a very slight thinning of crops indicating the need for proper drainage and the reduction of the excess of alkalinity.

In 1893 the Director of the Agricultural Experiment Station at the University of California was asked to investigate the agricultural possibilities of the land in the Imperial Valley. At that time it was proposed to provide an expedition properly equipped in order that the Director, Professor E. W. Hilgard, the great American authority on soils, might explore the region personally. The financial difficulties of the Company prevented carrying out the plan at the time, but a few samples of water from the lakes and of soil taken superficially, proved that the latter were very similar to that of the immediate bottom of the Colorado River, which previous analyses had shown to be of extraordinary intrinsic fertility.* In 1896 and 1897, some additional samples of soil and water were sent for examination. These corroborated the previous conclusions, but showed that a considerable quantity of alkali salts was present in the soils as well as in the waters, and thus indicated the desirability of a thorough examination of the region, from the soil standpoint. The subsequent soil investigations in the Imperial Valley and their effect on the fortunes of the region will be considered later.

Drainage.—Though the country as a whole lies ideally for irrigation and ordinary irrigation water drainage, the natural waterways are so far apart and so small and ill defined as to make the construction of an efficient, comprehensive drainage system almost as difficult and expensive as the irrigation canals. In the engineer's report to the Colorado River Irrigation Company, it was stated that the construction of a drainage system (though almost as expensive as the proposed

* Report, Agricultural Experiment Station, University of California, 1882.

rrigation system) was essential, but, some years later, when the work o be done was trimmed to the lowest practical minimum, it was ecided that a general drainage system was not immediately necessary nd possibly might never be required. This latter opinion was not s radical as might at first be assumed, because, even to-day, there are robably not more than 5 miles of drainage ditches in the valley. t is being realized in a general way that at some time provision for rainage must be begun, and within the next few decades doubtless a airly comprehensive plan will be developed. The diversion of the Colorado into the Salton Sea in 1905-06 resulted in eroding the beds f the Alamo and New Rivers into deep wide channels which will be he controlling features in the design of the ultimate drainage system or the valley, and thus produce a benefit which in the end must ertainly exceed the total damages resulting from such diversion.

Climate.—The climate of the region, with its long, hot, dry summers, is peculiarly favorable to agricultural luxuriance. Thus it is hat here the very earliest grapes, fruits, and vegetables are produced or the United States market, with the consequent advantage of ommanding the highest prices. This is notably true of the Imperial Valley cantaloupe, now famous all over this country, and of the early grapes, asparagus, etc. On account of the very low humidity and gentle winds which blow much of the time in hot weather, the sensible emperature—which is indicated by the wet-bulb readings and gives he measure of heat felt by the human body—is much less than the ctual temperature as measured by the dry bulb. It is conservative o say that a temperature of 110° in Imperial Valley is not more uncomfortable than 95° in Los Angeles or 85° in the more humid sections f the Eastern States. Furthermore, the nights are always cool, the ow humidity resulting in rapid and large daily temperature variations.

At the same time, the heat in the Colorado Desert and at Yuma vas proverbial, and one of the difficulties which the project had to encounter was the supposedly frightfully hot summers; indeed, the project would otherwise have been financed very much earlier. Since the control of the diversion canal was lost in 1905, the impression has become general that the project of irrigating this region was rejected by capitalists as involving too great engineering risks. As a matter of fact, he chief difficulty was the fear that the torrid climate would render colonization very difficult. It was for this reason only that Mr. George

Chaffey declined to consider Dr. Wozencraft's solicitations to undertake the enterprise as early as 1885, and it was Mr. Chaffey's subsequent experience in successfully establishing an irrigation enterprise in the interior of Australia, where a maximum temperature of 125° Fahr. was often reached, which led him to undertake the work here in 1900.

The International Boundary Line at the Sand Hills.—Perhaps, everything considered, the location of the International Boundary Line and the Sand Hills which lie to the west of Pilot Knob and overlap into Mexico for several miles, constitute the most important features of the irrigation, and protection from inundation, of the Imperial Valley. It is this which makes it impossible for the people of American Imperial Valley to organize to protect themselves under the laws of California. The menace is entirely on Mexican territory, and, apart from the difficulty of dealing with the problem as one of engineering and statecraft, is the worst feature of all, namely, the seeming injustice of compelling American citizens to protect their homes against a menace originating entirely on foreign soil.

Aside from the danger of the diversion of the Colorado to the west and into the Salton Basin, the important result of the present location of the International Boundary Line is that, practically speaking, water cannot be taken from the Colorado River and carried in canals lying wholly on American soil to the areas in American Imperial Valley susceptible of irrigation by gravity. It could be done, but it would require approximately 12 miles of a closed conduit running under the sand hills and costing at least $10 000 000, a sum practically prohibitive.

Water Rights.—Due to the divided authority of the National and State Governments with respect to permission for taking water from the Colorado River as a navigable stream, water appropriation notices then, as now, had to be posted and filed, under the laws of the State of California, and arrangements had to be made with the United States War Department as well, if such diversion interfered with navigation. It appears that no attempt was made to obtain permission from the War Department for taking water from the river, because it was almost impossible to cause any "interference with navigation." This failure to secure permission from the War Department, however, had a very serious result later.

Ideal Plans.—The ideal way to carry such a project through is now quite obvious. All the engineering features should have been carefully worked out, elaborate soil surveys should have been made by well-recognized authorities, and experimental farms should have been established. The irrigation system should have been built in sections and colonized before additional areas were covered by canals. Water rights, entirely free from any possibility of attack, should have been obtained. In the light of experience, the writer believes that, by all means, these should have been obtained from the Mexican Government, and the diversion should have been made on Mexican soil, or the development should have been made under the Carey Act.

Dealings with Mexico would have meant the abandonment of the idea of diversion works founded on solid rock, but a structure with a wooden caisson foundation extending under the gates proper and the wing-walls would have been just as safe as the concrete head-gate actually put in later, and would have cost little more money, if indeed as much.

The ownership of all private interests in the Salton Sink ought to have been acquired, and such permission obtained from proper Government authorities that this naturally depressed basin would ever be available without question as a receptacle for the seepage, drainage, and waste water from the irrigated lands and canals. Data as to silt deposition and the cost of removing it from canals and intakes should have been obtained from experiments carried out on a commercial scale. Various details of the project, in short, should have been worked out very carefully and adhered to.

However, as in many irrigation and other projects in the West, the garment had to be cut according to the cloth. The sum total of events resulted in carrying out the project along lines which were far from ideal, but which later proved to be possible of execution with a remarkably small amount of money, everything considered.

The California Development Company.

The first practical step toward the actual irrigation of Imperial Valley was the incorporation of the California Development Company, under the laws of New Jersey, on April 26th, 1896. After two years of vain endeavor to obtain permission from the Mexican Government for the American corporation to hold land and acquire rights

of way for the main canals into American Imperial Valley, it was found necessary to form also a Mexican corporation. The California Development Company has a capital stock of $1 250 000, divided into 12 500 shares of $100 each; the Mexican Company—*La Sociedad de Riego y Terrenos de la Baja California, Sociedad Anonima*—has a capital stock of $62 500, all of which is owned by the California Development Company. Hereafter in this paper the California Development Company will be referred to as the C. D. Co., and the subsidiary Mexican corporation as the Mexican Co.

The general practice throughout the West was, and still is, the sale of the "water right" to settlers at a definite price per acre—usually the right to buy water thereafter at specified prices. The arrangement adopted in this case was the formation of mutual water companies which would receive water wholesale and distribute it to their stockholders, the capital stock of such mutual companies constituting the water right.

Organization Under the Carey Act.—It would undoubtedly have been much better if the desert land in the United States had been segregated, and if the project, as far as American territory was concerned, had been carried out under the Carey Act. This Act, however, had not been passed when the original investigations were made, and, when financial arrangements were concluded, the California Legislature had adjourned and would not meet for nearly two years. Such delay was deemed too great.

Water Appropriations.—Water filings were made on April 25th, 1899, on the right bank of the Colorado River about 3 000 ft. above the International Boundary Line, by Mr. C. N. Perry, on behalf of the C. D. Co., appropriating 10 000 sec-ft., of the flow of the Colorado River to be used for the irrigation of American lands in the Imperial Valley. No serious attempt was made to obtain water rights in Mexico—in Mexican territory there was no chance to found diversion works on rock, and money for the first work of promotion would have been difficult to obtain with a projected intake in that country.

Rights of Way.—The C. D. Co. purchased 316 acres of patented land along the river just north of the International Boundary Line. and these included the rocky point of Pilot Knob; and the Mexican Co. acquired 10 000 acres in Mexico, belonging to Gen. Guillermo Andrade, and lying generally south of the Boundary Line, as shown

in Fig. 9, together with the bed of the Alamo River, which extended beyond the boundaries of this tract. In the American Imperial Valley (all the land belonging to the Government except Sections 16 and 36, belonging to the State of California, and known as "school sections") rights of way could not be purchased outright, but easements therefor were easily obtained as at present by application to the Secretary of the Interior, accompanied by maps and descriptions of the proposed constructions. All rights of way and property required for the construction of the project were thus arranged.

Contractual Relation of the C. D. Co. and the Mexican Co.—The two companies entered into a contract by the terms of which the C. D. Co. turned over to the Mexican Co. all the water to be diverted from the Colorado River by the former where the canal crosses the International Boundary Line at Algodones; the Mexican Co. agreed to deliver water to water users in Mexican territory as required and the remainder of the supply—the larger part by far—to the American water users at points on the International Boundary Line from 40 to 50 miles west of the river, and, from the water users of both countries, to collect for the water furnished, on a quantity basis; the C. D. Co. agreed to build, maintain, and operate all the Mexican Co.'s irrigation construction in Mexico; the Mexican Co., in consideration thereof, agreed to pay the C. D. Co. all sums received by the former for water rights, water stock, and water rentals from water users in the United States. These agreements were limited to water for lands which were irrigable by gravity from the system of canals beginning at the head-works constructed. It was stipulated, further, that no contract should be entered into with the Mexican Co. giving any person or corporation superior right over any other water user by reason of priority in date of contract or otherwise, and that the C. D. Co. should not be responsible for failure to deliver water to the Mexican Co. from any cause beyond its control, although admitting obligation to use due diligence in protecting canals and maintaining the flow of water therein.

By this arrangement, the Mexican Co. retains the money received from the water delivered to Mexican water users, and is put to no construction, maintenance, or operation expense whatever. This arrangement, however, is not as advantageous as at first appears, because the gross annual water rentals from Mexican water users did

not amount to $10 000 gold per annum until the beginning of the ninth year, while the right of way contains at least 2 500 acres of land and includes 50 miles of the Alamo River channel, which is utilized as a main canal. It will be a number of years yet before the receipts of the Mexican Co. will be sufficiently large to make the contract an unusually profitable one.

Mutual Water Companies.—Next to the general plan of arranging to require the purchase by settlers of the water right usual in such cases, the fundamental idea was delivery of water to mutual water companies instead of individuals, the mutual companies to be operated by the holders of stock, namely, the farmers in their respective districts. The various mutual companies thus run their own local affairs and join together, through the C. D. Co. and the Mexican Co., in a community main canal leading from the river to the settlement west of the Sand Hills.

Triparty Contracts.—The mutual water companies required the construction of a distribution system, and ought or ought not to have paid a bonus for the contract to receive water at the International Boundary Line, depending entirely on the conditions under which the water should be delivered and the price to be paid for it. A triparty contract was entered into between the Mexican Co., the C. D. Co., and each of the mutual water companies, under the terms of which the Mexican Co. agreed to supply water to the mutual water companies "on demand" and at definite points on the International Boundary Line in the Imperial Valley for 50 cents per acre-ft., to be used only on lands within the respective districts; provided, however, that the aggregate quantity of water necessary to deliver under the contract should not exceed four times the number of acre-feet per annum that there were shares of capital stock in the mutual company; the mutual company agreed to order and pay for at least 1 acre-ft. of water each year for each share of its stock sold and located, regardless of its use by the mutual company or by its stockholders; the C. D. Co. agreed to build the distribution system of the mutual company and to maintain certain definite portions of the canal thereof perpetually, reserving the right to develop and use the water-power that might be obtained from the waters running through any of the canals, including those of the mutual company; a provision was made that at the end of 3 years the loss of water to the C. D. Co. in evaporation from

the canals of the mutual company should be determined, and such an extra allowance of water be supplied, as so determined, to the end that only the net quantity reaching each half section of land should be paid for; and the mutual water company turned over all its capital stock to the C. D. Co. and agreed to locate such stock on any lands within the exterior boundary lines of its district on order of the C. D. Co. The C. D. Co. sold the capital stock of these various mutual companies to the settlers, and with the proceeds built the main canals in the United States, the canal system in Mexico which belongs to the Mexican Co., and the distribution systems which became the properties of the various mutual water companies.

There were eight of these triparty contracts; they were essentially similar, though no two were exactly alike in every detail. The contract between the Mexican Co. and the C. D. Co., and the triparty contract as just outlined, together with the by-laws of the mutual companies, show the contractual relation of the water user to the organizations on which he depends for water. These by-laws, in general, provide that each share of stock shall represent the right to purchase water for the irrigation of 1 acre of land; that stock issued shall have written on its face a description of the land on which it is located; that no stock shall be located on any lands outside those described in the articles of incorporation; that one share and no more shall be located on each acre of land which can be served by the ditches of the company; that owners of stock issued but not located shall not be entitled to receive any water represented thereby, but shall, nevertheless, be liable for all assessments, the same as other outstanding stock of the company; that the shares may be transferred; and that acceptance by any stockholder of a certificate of stock shall be considered as a ratification by him of any and all contracts between the mutual company in question and the C. D. Co.

The inter-relations of the water users and the various corporations have been given in detail because of a general impression that the plan was devised for the purpose of taking advantage of the settlers. In its operations it has resulted in no unfairness of any importance to any of the parties concerned. Considering all the circumstances, the prices charged for water rights were very low—$5.75 at the beginning, up to $20 at present, and averaging $12 per acre as the total cost to the settler, on easy terms—and the total annual water rental from the

TABLE 7.—Comparative Statement of Earnings and Expenses of the California Development Company, for November, 1909.

(Property on a seriously deteriorating basis.)

	November, 1909.	5 months ending November 30th, 1909.
Earnings:		
Water sales	$13 906.20	$93 296.75
Water-power royalties	418.90	1 772.80
Rent, buildings and other property	82.05	488.51
Miscellaneous earnings	66.45	420.68
Gross earnings from operation	$14 473.60	$95 918.74
Operating Expenses:		
Maintenance, canals and structures:		
Superintendence	$163.83	$641.63
Maintenance and cleaning canals	10 588.76	47 130.74
Bridges	223.04	408.88
Canal structures	901.70	2 444.57
Buildings, fixtures and grounds	824.81	3 192.44
Total	$12 702.14	$53 818.26
Maintenance of levees:		
Superintendence	.....	$210.00
Patroling	$102.55	377.87
Roadway and track	126.12	1 034.01
Telephone and telegraph lines	84.87	217.04
Buildings, fixtures and grounds	.....	44.37
Total	$313.54	$1 883.29
Maintenance of equipment:		
Vehicles	$85.23	$576.47
Grading implements	.....	14.23
Corrals	29.01	525.56
Machinery	.....	533.45
Shops	34.71	155.25
Automobile	135.36	1 115.04
Motor cars	83.34	154.46
Dredges	281.42	938.18
Total	$649.07	$4 012.64
Distribution of water:		
Superintendence	$163.70	$997.22
Zanjeros	760.00	3 858.63
Calibration and water measurement	21.52	365.00
Telephone and telegraph lines	163.40	815.53
Damages	.....	50.00
Total	$1 108.62	$6 086.38
General expense:		
Salaries and expenses, general offices	$2 981.66	$15 444.09
Office expenses	332.37	1 515.04
Law expenses	618.28	3 899.92
Stationery and printing	132.36	684.86
Other expenses	107.40	459.67
Total	$4 172.07	$22 003.58
Total operating expenses	$18 945.44	$87 804.15
Net earnings	*$4 471.84*	$8 114.59
Taxes	$197.76	$1 134.88

water users in the valley will not suffice to pay maintenance, operation, and general expenses, properly figured, until such time as about 700 000 acre-ft. of water are sold annually. At the end of 9 years the sales have not yet reached that figure. Fig. 4 shows the boundaries of the lands of the various mutual water companies in the valley and under whose distribution systems lie all the lands which are as yet irrigated.

Operation of Triparty Contract.—For 3½ years the writer was General Manager for both the C. D. Co. and the Mexican Co., and handled all matters between these companies and the various mutual water companies. During the latter portion of that time, the protection of the Imperial Valley from inundation by the Colorado had become quite as important as its irrigation, and, for this protection, of course, no provision was contemplated in these contracts. Except for that, the arrangement proved to be very satisfactory, and produced a smooth and comfortable relationship unusual in irrigation enterprises. As a result of litigation, the Supreme Court of California has just declared the whole scheme practically legal. The Imperial Irrigation District was created several months ago, and the directors thereof have decided to take over only the functions which the C. D. Co. and the Mexican Co. now perform, and will not interfere in any way with the mutual water company plan of organization, or the water companies themselves.

TABLE 8.—Statement of Earnings and Expenses of La Sociedad de Irrigacion y Terrenos de la Baja California.

	November 5th, 1909.	5 months, ending November 30th, 1909.
Gross Earnings	$641.20	$5 127.39
Operating Expenses:		
Distribution of water	0.00	0.00
General expense:		
Salaries and expenses, general officers and clerks	421.54	2 073.85
Office expenses	64.07	341.32
Law expenses	223.99	696.64
Stationery and printing	39.55	101.21
Inspection fund (Mexican Government)	150.00	750.00
Other expenses	231.25	663.47
Total	$1 130.40	$4 626.49
Total operating expenses	$1 130.40	$4 626.49
Net earnings	*$489.20*	$500.90

Imperial Land Company.—The parties who were induced to back the enterprise financially were afraid of the colonization end, and would have nothing whatever to do with it. Accordingly, it was necessary to form a colonization company—the Imperial Land Company—which was incorporated under the laws of California in March, 1900, and consisted in part of some of the promoters of the C. D. Co. and in part of other people. This corporation contracted to do all advertising and colonizing and sell all water stock in consideration of having the exclusive privilege of town sites and a commission of 25% on water stock sales. By using Government land scrip, this company obtained immediate ownership in fee simple of tracts of land in various parts of the valley and subdivided them into town sites. These town sites were covered with water stock in order to obtain water for domestic and municipal use through the assistance of the mutual companies, because no wells, except some very deep and unsatisfactory ones quite recently sunk on the east side of Imperial Valley, have ever been possible for domestic supply. The Imperial Land Company thus established the town sites of Mexicali, in Mexico, and Calexico, Heber, Imperial, and Brawley, in California. The other town sites—El Centro, the county seat, Holtville, Seeley, Dixieland, and several smaller places were platted and put on the market by other parties later.

TABLE 9.—Operating Expenses of California Development Co., January 1st, 1908, to March 31st, 1909.

	1908. 12 months.	1909. January, February, and March.
Maintenance, canals and structures	$71 419.94	$18 177.32
Maintenance, levee	10 260.35	647.24
Maintenance, equipment	18 528.69	4 182.21
Distribution of water	15 613.42	4 559.10
General expense*	75 162.82	12 277.76
Construction of canals	73 765.12	27 359.47
Construction of levees	32 303.09	32 297.84
Totals	$297 053.43	$99 500.94

* Of this sum, $30 665.28 was litigation expenses and costs.

During 1911 the total net deliveries of water to the mutual water companies in the United States were 597 178 acre-ft., or $298 490.98.

This colonization company in general was successful, but not to the extent which would be expected, considering the unprecedentedly rapid settlement of the region, and the contract was certainly a fair one to

the C. D. Co., up to the time of its abrogation in 1906. Water stock was sold to the settlers for small cash payments and notes payable in five yearly settlements at 6% interest, such notes being secured by a pledge of the water stock purchased. Many of the settlers had scant means and only a filing right on the land, so that the water stock was not made appurtenant to the land, but left as personal property. The initial payment went to the Imperial Land Company, and was by it used for advertising and other essential purposes, the collateral notes secured by the water stock being taken by the C. D. Co.

TABLE 10.—Average Diversion and Deliveries of Water by the Canal Systems of the C. D. Co. and the Mexican Co. for the Week Ending January 19th, 1912.

Gauge at Yuma		15.3 ft.
Gauge opposite intake		105.9 "
Elevation of bottom of diversion gate		98.0 "
Average flow of Colorado River at Yuma		4 000 sec-ft.
Diversion from Colorado River at Andrade		1 559 " "
Used in Mexico	37 sec-ft.	
Used in United States	894.6 " "	
*Wasted at Rositas waste-gate	321.3 " "	
Total		1 252.9 sec-ft.
Total loss, Andrade to Sharps		306.1 sec-ft.

* 171.1 sec-ft. of this passed through the plant of the Holton Power Company *en route* to this waste-gate for developing electrical energy.

This loss equals 19.6% in about 46 miles of main canal, chiefly the bed of the old Alamo River, or 0.43% per mile on the average—an extremely low figure.

Management of the C. D. Co.—Delta Investment Company.—Until the water rentals became of importance, these collateral notes constituted the only receipts of the C. D. Co., and these assets were looked on with considerable suspicion by the financial institutions of Los Angeles. Nevertheless, they might have been taken as collateral at about 25 cents on the dollar had not the merit of the entire enterprise been rendered questionable in various ways, as explained later. When this occurred the Delta Investment Company was formed—in the fall of 1901—with assets consisting solely of C. D. Co. and Imperial Land Company stock contributed by the wealthier people of the enterprise. This company was given a contract to take over all the C. D. Co.'s bonds at 50 cents on the dollar, and all its collateral notes and mortgages at the same discount. By this arrangement, the Delta Investment Company practically controlled the C. D. Co., although the amount of the C. D. Co. stock held by it was much less than a majority.

TABLE 11.—Annual Expenditures of Imperial Water Company No. 1 for 1911.

Capital stock = 100 000 shares, all of which have been sold, and are located on 100 000 acres of land. Total length of canals = 373.25 miles.

Maintenance:		
Superintendence	$7 000.00	
Engineering	1 600.00	
Corral	3 805.11	
Automobile	500.00	
Shops	2 463.13	
Materials and supplies	22 046.11	
Labor, men and teams	75 887.41	
Damages	645.91	
Muskrats, bounty at $1 each	492.00	
		$114 439.67
Operation:		
Superintendence	$3 815.74	
Engineering	168.81	
Zanjeros	22 609.02	
Corral	3 192.36	
Automobile	476.39	
Materials and supplies	1 892.98	
Telephone	260.16	
Water meters	260.47	
		32 675.93
General Expense:		
Salaries	$6 410.02	
General expenses	2 971.13	
Printing and stationery	439.94	
Taxes and insurance	956.59	
Furniture and fixtures	520.45	
Legal expenses	10 889.38	
		22 187.51
Imperial Water Company No. 1, expense		$169 303.11
Water Bought (from the C. D. Co.) 305 183 acre-ft., less 10% allowance for seepage and evaporation, at 50 cents per acre-ft., on net amounts		137 332.50
*Total expenditures**		$306 635.61

*The expenses of the company were almost exactly $1.70 per acre, and the water rentals paid to the C. D. Co. $1.37 per acre. The total cost to the farmers, therefore, averaged $3.05 divided by 2.747, or $1.11 per acre-ft.—a very low figure for water in California, where the "water right" averages $12 per acre, or indeed much more.

It must be admitted that the Delta Investment Company took over such securities at a larger price than could have been obtained from any other source. Nevertheless, the securities were really good, everything considered, and quite a few large and apparently strange and dishonest transactions were made between the two corporations. Money was forthcoming for construction purposes, but was costing the C. D. Co. $2 for every $1 obtained. The result was that in a couple of months serious dissensions arose, and in February, 1902, an adjustment was made cancelling the contracts with the Delta Investment Company and Mr. Chaffey, who thereupon retired from the enterprise. March 1st, 1902, therefore, found the C. D. Co. a going concern, and Imperial Valley a reality, but the parent company, with all its bonds gone, its

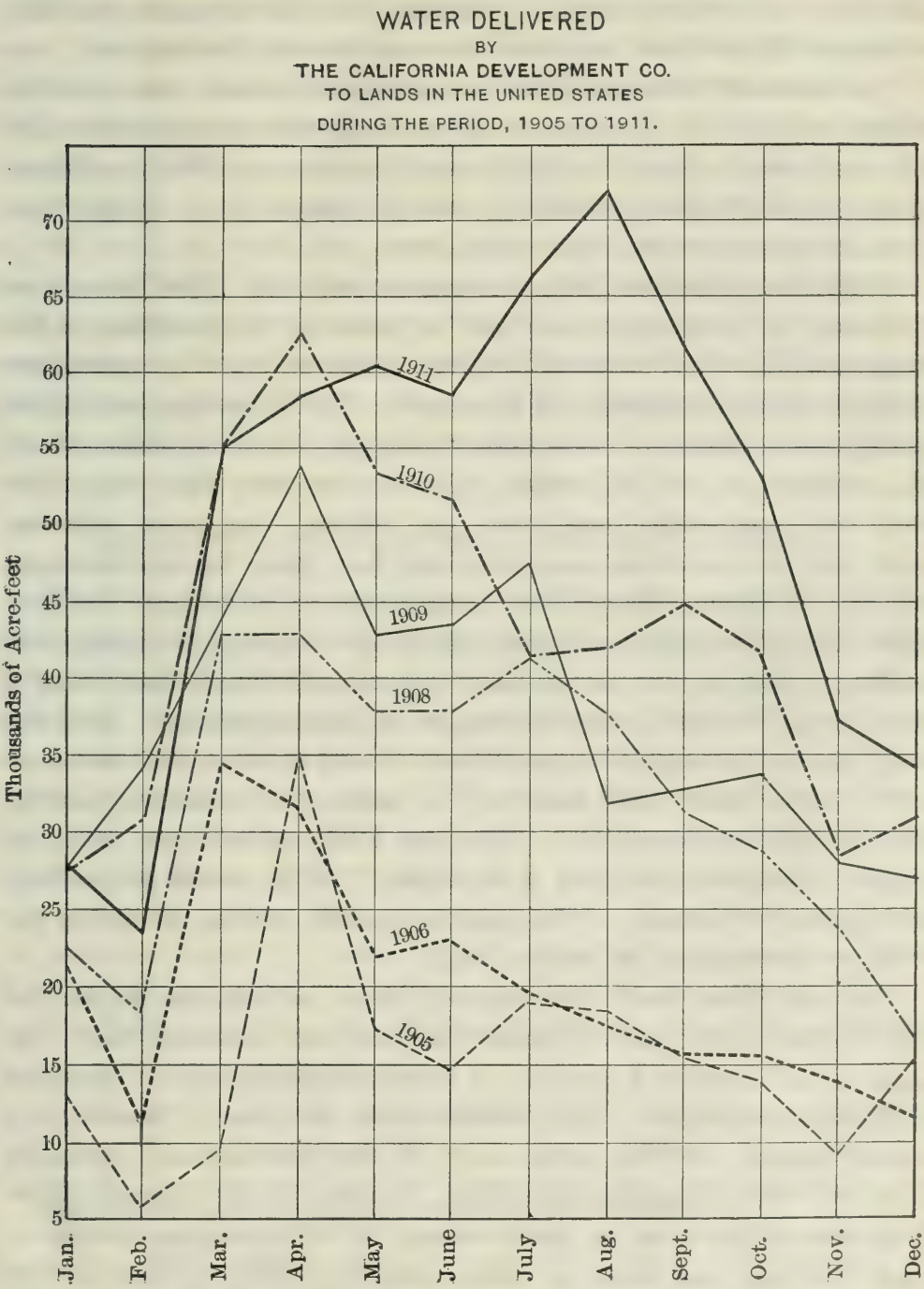
WATER DELIVERED
BY
THE CALIFORNIA DEVELOPMENT CO.
TO LANDS IN THE UNITED STATES
DURING THE PERIOD, 1905 TO 1911.
Thousands of Acre-feet
70
65
60
55
50
45
40
35
30
25
20
15
10
5
1911
1910
1909
1908
1906
1905
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.

FIG. 12.

collateral notes and mortgages largely depleted, no money in the treasury, and deeply in debt. Shortly afterward actual results from farming under the project were so reassuring that the Company was able to borrow $25 000 from the First National Bank of Los Angeles.

The contract with the Delta Investment Company was a serious thing for the C. D. Co., but, to be perfectly fair in presentation, it must be borne in mind that the financial interests had their confidence in the project violently shaken by advance rumors of an adverse Government soil report (to be discussed later).

With the exception of the arrangement with the Delta Investment Company, no proper criticism can be made of the handling of the finances of the whole irrigation project, as far as any of the promoters of the irrigation company are concerned. The writer has had opportunity and occasion to investigate thoroughly the relationship of all the corporations, and in common fairness must state that, though the deals back and forth were many and diverse, they were otherwise with very few exceptions reasonable and fair, when the circumstances and reasons which produced them are given the proper weight. Furthermore, the general aims and plans which the company practically succeeded in carrying out do not merit any more criticism than those of the average Western irrigation project, if indeed as much. Had the break in the Colorado River never been allowed to get beyond control—and it never would have happened, in spite of all obstacles, had the loan of the Southern Pacific Company (referred to later) been arranged 6 months earlier than it was—the C. D. Co. would undoubtedly have proved to be one of the most successful private irrigation enterprises throughout the entire land.

Colorado River Land Company.—It is well at this time to mention the Colorado River Land Company and the New Liverpool Salt Company. The former is a corporation consisting principally of Southern California stockholders, incorporated under the laws of Mexico, and owning about 1 000 000 acres south of the International Boundary Line and west of the Colorado River. It owns all the Colorado River Delta west of the river in Mexico except 162 000 acres, the location of these holdings and those of other important Mexican land owners being shown on Fig. 4. The existence and operation of this corporation have lately become important as being the agency through which the United States Government has handled the river control work

recently done by it. The company will hereafter be referred to as the C. M. Co., as it is locally called.

The New Liverpool Salt Company.—This corporation was organized many years ago for the purpose of obtaining salt from the deposits in the bottom of the Salton Sink, and began operations in 1884. In 1904 its plant was reasonably satisfactory in its details and had a capacity of 1 200 tons of salt per month. The actual value of the plant and the salt beds, taking into consideration the excellent quality of the salt,* the conditions under which the Company operated, and the competition it had to meet, is of course impossible to determine without access to the company's records. It appears, however, that negotiations at that time were pending for its sale, the figures being $150 000 asked and $100 000 offered. When the water began to come into the sink in large quantities, negotiations were dropped, and the entire plant was soon buried by the Salton Sea.

Operations of the California Development Company.

When the C. D. Co. was ready to begin operations, there was on the lower river a dipper dredge with a 4-yd. bucket which had been built and equipped by the Hon. Eugene S. Ives, of Yuma, Ariz., and his associates, for digging irrigating canals near Yuma. This dredge was bought by the company in exchange for guaranteed bonds, floated down the river, and, in August, 1900, set to work excavating a canal along the lines marked "Original Intake" in Fig. 13 and then following the old Alamo overflow channel to a point 8 miles below. From that point the Alamo channel, with a little diking here and there, had sufficient capacity to carry for some time the water needed. From the beginning of actual construction until he left the enterprise (April, 1900, to February, 1902), Mr. George Chaffey was President and Chief Engineer of the Company.

About 500 ft. above the Boundary Line a temporary wooden head-gate, Fig. 11, known locally as the "Chaffey" gate, was put

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* The published analyses of the deposit give the following average:

Sodium chloride	96.15
Sodium sulphate	0.70
Calcium sulphate	0.60
Magnesium sulphate	1.60
Insoluble	0.10
Water	0.85
	100.00

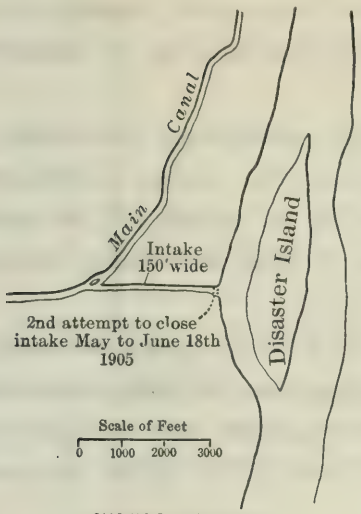
The California State Mineralogist reports the value as $1 per ton.

in. This was a well-designed and well-built wooden, A-frame, flash-board gate, 70 ft. long, 15 ft. high, with a plank floor, and founded on piling. When it was built* nothing was known or even suspected with reference to the rapid and large variation in elevation of the river bed at varying flood stages and otherwise, and it is not surprising, therefore, that the floor was not put as low as it should have been, but, even so, it was not as deep as planned on account of an unnaturally early summer flood. The structure was made no larger, not because of cost, but because it seemed certain that when more water than the gate's capacity should be required, that fact would mean such revenues as to permit building the permanent concrete and steel diversion works at Pilot Knob, regardless of all other considerations. In passing, it may be said that the construction and design of this temporary head-gate was fully equal to that of any throughout the West, even to-day. The floor, however, was quite too high.

At what is known as Sharp's Heading, the Alamo channel was abandoned as the main canal, and the controlling works for the valley end were put in. These consisted of a wooden, A-frame, flash-board gate in the continuation of the Alamo, a similar gate at the head of the Encina or West Side Main canal, and a combined gate and drop, known as Sharp's Head-gate, from which leads off the Central Main, the chief canal in the valley.

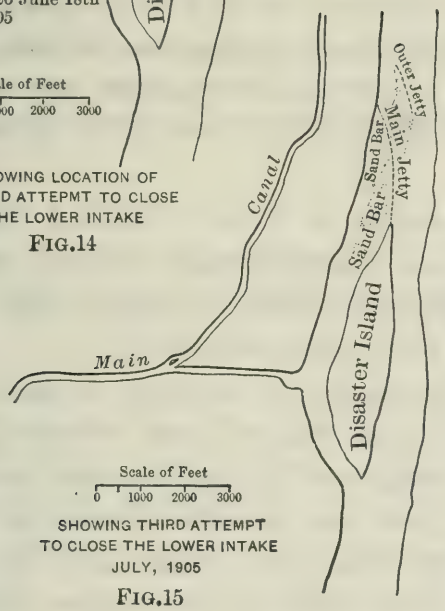
This last structure is well worth describing in some detail. In the first place, it is a most vital part of the system, because, being a combination of a drop and regulating gate, were it to fail, the water in the Alamo or Main Canal above it would immediately be lowered far too much to permit taking out any whatever for the East and West Side Mains. To realize the consequences of this, it must be remembered that irrigation water is needed every day in the year, and that no stock and domestic water for the entire region, except for the Town of Holtville, can be had, except from the irrigation system and by being brought in by the railroad in water cars. In the second place, for several months consecutively, in each year since 1905, it has been taxed beyond the capacity for which it was designed, without developing any serious weakness. Furthermore, at intervals of about 18 months, since it was put in service in 1903, the canal above it has

* Nothing really was known about the changes in elevation of the river bed until 1907.



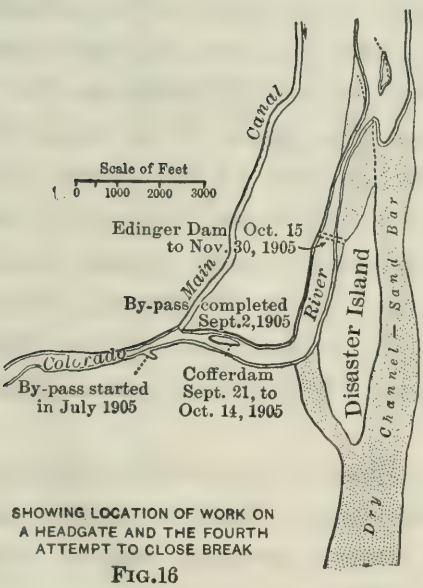
SHOWING LOCATION OF SECOND ATTEPMT TO CLOSE THE LOWER INTAKE

FIG.14



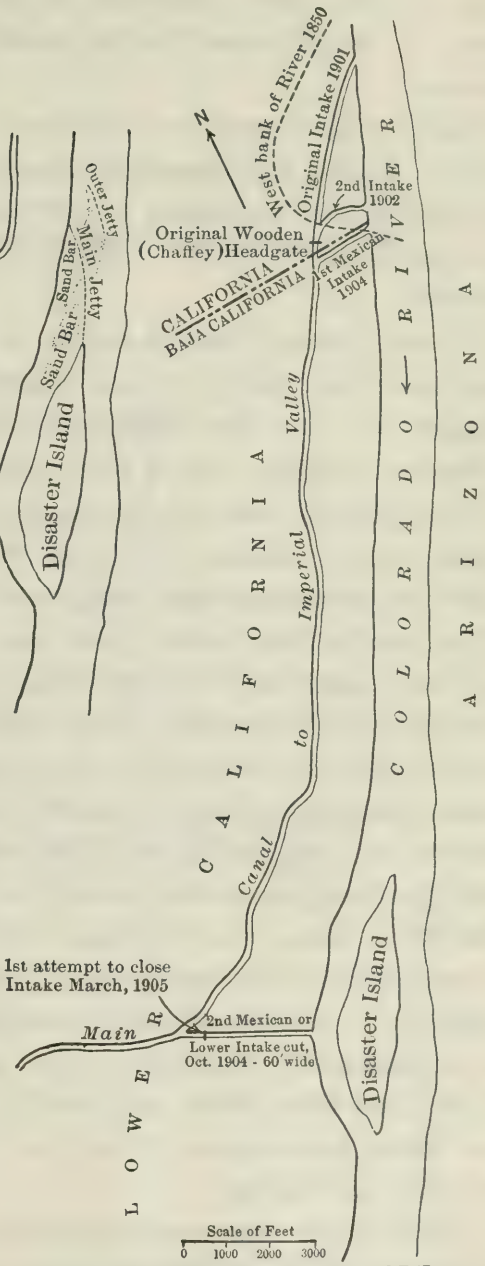
SHOWING THIRD ATTEMPT TO CLOSE THE LOWER INTAKE JULY, 1905

FIG.15



SHOWING LOCATION OF WORK ON A HEADGATE AND THE FOURTH ATTEMPT TO CLOSE BREAK

FIG.16



MAP SHOWING VARIOUS INTAKES OF THE CANAL TO IMPERIAL VALLEY AND LOCATION OF THE FIRST ATTEMPT TO CLOSE THE LOWER ONE

FIG.13

been emptied for periods of not more than 60 hours to permit of inspection and light repairs, but the very first overhauling or extensive repairs were begun on January 5th, 1912.

The writer confesses to a predilection for permanent structures of masonry, concrete, or steel, and this gate and the Alamo Waste-gate, built in 1905, were nightmares to him while in charge of the properties. It would seem that a large part of this was worry wasted, however.

Sharp's Head-gate was designed by, and built under the direction of, Mr. C. N. Perry, then Resident Engineer of both companies, the fundamental idea being to cut up the foundation into a number of water-tight compartments. Plates XLIV and XLV show this construction.

Where Beltran's Slough leaves the Alamo channel, a wooden, flashboard gate was built to waste water through Beltran's and Garza's Sloughs into the New River, but about 3 months after being put in service it failed, due to back currents below it.

The original plan for supplying the territory to the east of the Alamo was to utilize the Alamo channel from Sharp's Heading to Holtville, an earthen dam being used to bring the water to the surface of the land at that point. This dam soon failed, and the canal from there was connected with the Alamo at a point about 1½ miles above Sharp's, such connection being made in record time, with a cross-section only large enough for the demand. The idea was that erosion would enlarge it, which in general has been the case, although some blasting was required to assist the action. Originally known as No. 5 Main, the canal is generally called the East Side Main. It, as well as the West Side Main, is occasionally broken in places by the severe rainstorms—almost cloudbursts—which occur at infrequent intervals in the region. To provide absolute protection against such damage would be very expensive, and neither No. 5, which owns the exposed portion of the East Side Main, nor the C. D. Co., which owns all the West Side Main, has done so. Otherwise, they, as well as the Central Main, are quite satisfactory.

Main canals were constructed from Sharp's to serve the territory between the New and Alamo Rivers (the Central Main); a second, the West Side Main, crossed New River to serve territory west of that waterway, and a third, the East Side Main, to serve the territory

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PLATE XLIV.
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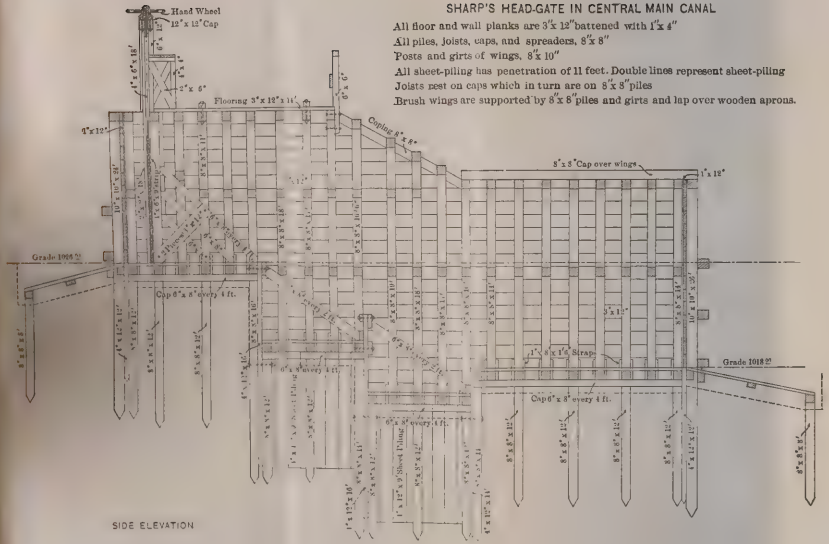
SHARP'S HEAD-GATE IN CENTRAL MAIN CANAL

All floor and wall planks are 3″ x 12″ battened with 1″ x 4″
All piles, joists, caps, and spreaders, 8″ x 8″
Posts and girts of wings, 8″ x 10″
All sheet-piling has penetration of 11 feet. Double lines represent sheet-piling
Joists rest on caps which in turn are on 8″ x 8″ piles
Brush wings are supported by 8″ x 8″ piles and girts and lap over wooden aprons.


Hand Wheel
12″ x 12″ Cap
Flooring 3″ x 12″ x 11′
Coping 8″ x 8″
8″ x 8″ Cap over wings
1″ x 12″
Grade 1004.5′
Cap 6″ x 8″ every 4 ft.
3″ x 12″
1″ x 6″ x 1′6″ Strap
Grade 1018.2′
Cap 6″ x 8″ every 4 ft.
6″ x 8″ every 4 ft.
6″ x 8″ every 4 ft.

SIDE ELEVATION

east of the Alamo. In 11 months, or in June, 1901, delivery of water was begun through the Boundary Canal as far west as Calexico, and the Central Main was put into service in March, 1902, or in 19 months.

Imperial Water Companies Nos. 1, 4, 5, 6, 7, and 8 were formed, and triparty contracts were entered into with each. The C. D. Co. constructed the distributing systems for these districts, with the exception of that of Imperial Water Co. No. 7.* The total length of canals in all these distributing systems was approximately 700 miles on January 1st, 1905, and there were also about 80 miles of canal belonging to the C. D. Co. and the Mexican Co., making the total about 780 miles. During 1905 and 1906 relatively little canal building was done, because the river got beyond control; and, from 1907 to 1911, inclusive, the increase has been less than 20%, on account of excessive litigation following the vast expenditures for controlling the river, and because the canals existing on January 1st, 1905, covered 85% of the territory now under ditches.

With the exception of a permanent diversion gate at the river, two permanent structures replacing temporary ones in the valley, the building of the Alamo Waste-gate (Fig. 27), just above Sharp's Heading (June 25th to August 17th, 1905), and another in the Central Main at Station 134 (November 13th, 1904, to January 12th, 1905), that portion of the canal system completed on January 1st, 1905, has not been essentially changed or enlarged, and, with few exceptions, the original structures are still being used. There is a marked tendency on the part of the mutual water companies to replace wooden structures with permanent ones of reinforced concrete, but otherwise in general the canal systems are as satisfactory as any which could be devised.

The irrigation service afforded to farmers in Imperial Valley is the best of which the writer has ever heard. This has been the case with the exception of three short periods: the winter of 1904, 1 month (November) in 1906, and a total of 2 months in 1910, when there were shortages of water. Indeed, so accustomed are the water users

* The water rights for all the land south and east of the district of Imperial Water Co. No. 5 which could be irrigated by gravity from what was known as the Holt Heading—where the East Side Main heads—approximately 18 000 acres, were sold for $50 000. The purchaser, Mr. W. F. Holt, formed Mutual Water Company No. 7, constructing the distribution system and selling for his own benefit all the capital stock of this company. The fact that this deal was made at the rate of $3 per acre, including the consideration for the proportional cost of the controlling works in the valley and of the main canal thereto from the Colorado River, for one of the very richest sections of land, shows plainly the financial straits of the company at that time.

of this region to obtaining all the water they want whenever they want it, that a suggestion of delivery in rotation—which is done in almost all irrigation projects—would doubtless meet violent opposition.

A preliminary summary, issued on December 15th, 1911, by the U. S. Census Bureau, states that, in 1909, 2 664 104* acres of land were irrigated in California, of which 220 000 acres, or one-twelfth, were in Imperial Valley. The percentage irrigated of the whole number of farms was 44.6, or 39 352 acres. The area included in projects completed and under construction was 5 490 360 acres, or slightly more than double the present irrigated area. Probably there will soon be 450 000 acres under the Imperial Valley canals, or just about the same proportion of one-twelfth. Of the acreage irrigated in 1909, there were 400 acres (0.01%) under the canals of the U. S. Reclamation Service; 3 490 acres (0.1%) under the U. S. Indian Service canals; 173 793 acres (6.5%) under canals of irrigation districts; 779 020 acres (29.2%) co-operative enterprises; 746 265 acres (28%) commercial enterprises; and 961 136 acres (36.1%) individual or partnership enterprises. Of the irrigated acreage in 1909, 71% was watered by works controlled by the water users. Of the remaining 29%, almost one-third is under the canals of the C. D. Co. Aside from the very large area covered by the canals of this project, its relative importance is vastly increased by the vital necessity for continuous service every day in the year, which has no counterpart of which the writer knows, and the minimum daily demand in winter is one-fourth of the maximum.

Obstacles Encountered by the C. D. Co.

The settlement of Imperial Valley† took place more rapidly than any of the men interested in the project had even hoped, and constituted the most marvelous achievement of irrigation in the West, up to that date at least, and probably to the present time. On January 1st, 1901, with the exception of a party of surveyors, not a single white

* Undoubtedly, the greater part of this total is irrigated only after a fashion, so that the relative importance of the irrigated area in Imperial Valley is much greater than the figures indicate.

† The Imperial Land Company decided to use the name "Imperial Valley," for the region to be covered by the irrigation canals, instead of "Colorado Desert" or "Salton Basin," partly to distinguish between the reclaimed and unreclaimed areas, but chiefly for the effect of the name on readers of the colonization literature put out by the company. The name, "Imperial Valley," is firmly established as referring to the cultivated portion of the Colorado Delta west of the river, whether north or south of the International Boundary Line

PLATE XLV.
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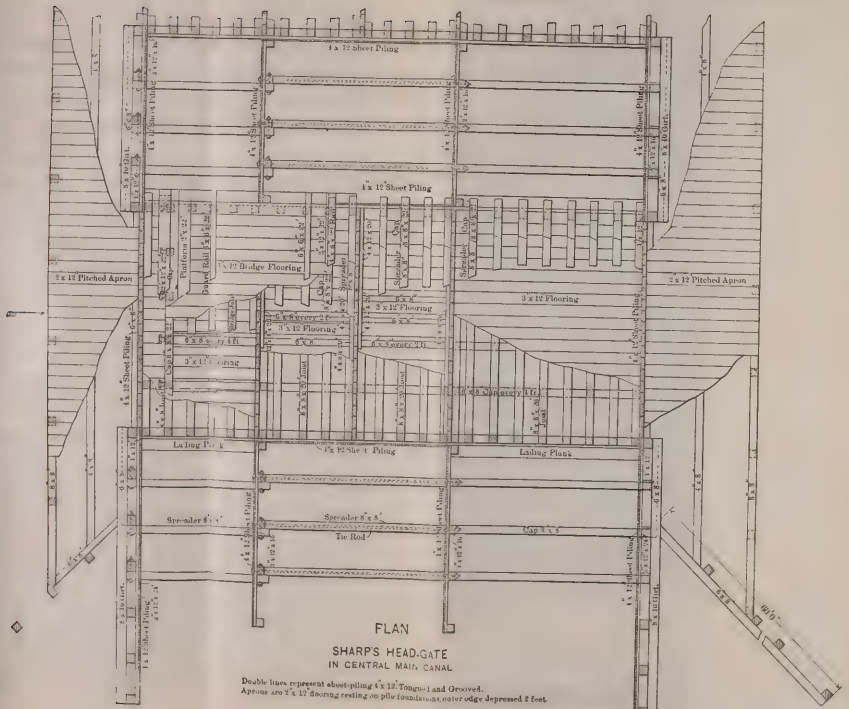

FLAN

SHARP'S HEAD-GATE
IN CENTRAL MAIN CANAL

Double lines represent sheet-piling 4" x 12" Tongued and Grooved.
Aprons are 3" x 12" flooring resting on pile foundations, outer edge depressed 2 feet.

man lived in the whole region; by January 1st, 1903, 2 000 people had come in; by January 1st, 1904, probably 7 000 people had made their homes in the new district, and by January 1st, 1905, the population was between 12 000 and 14 000. As early as 1904 there was a branch railroad through the district from the Southern Pacific main line at Old Beach, since called Imperial Junction, and, at the beginning of 1905, there were seven towns, with stores, banks, etc., 780 miles of canals, about 120 000 acres of land under actual cultivation, and 200 000 acres covered by water stock.

This unprecedented and unexpectedly rapid development overtaxed the resources of the C. D. Co., and, in addition, there were several untoward factors which accentuated the difficulty. These were serious complications in the United States Government Land Survey of the region, an extremely unfavorable soil report by the United States Agricultural Department, agitation for the United States Reclamation Service to supplant the irrigation system of the valley, a question as to the right to divert water from the Colorado River, and troubles at the intake by silt depositions.

United States Land Surveys in the Imperial Valley.—That portion of the Imperial Valley north of the 4th standard parallel was supposed to have been surveyed in 1854-56. The maps and notes for it were accepted, but there is at least some question whether the survey was ever actually made in the field. Later, in 1880, after the International Boundary Commission had surveyed the Boundary Line between the United States and Mexico and marked it continuously with permanent monuments, the area south of the 4th standard parallel was surveyed, this being locally known as the "Brunt" survey. The colonization company, in April, 1900, put out surveying parties under the direction of Mr. Perry, now County Surveyor of Imperial County, to re-run the Government lines and establish corners so that settlers might have proper descriptions for the tracts of land they wished to file on, and also that the distribution systems of the various mutual companies might be located along the Government subdivision lines, as the topography of the land is such that this ideal canal location is generally feasible. Mr. Perry found nearly all the corners of the Brunt survey, and used the notes showing certain connections made with the survey of 1856 along the 4th standard parallel. In this way

the lines to the north were retraced, but, some time later, when the survey had extended farther and the work of retracing the lines east of the Alamo River was commenced, it was discovered, by encountering natural features given by the notes of the 1856 survey, 2 miles or more out of correspondence, that there were serious errors. Exhaustive search was then made for the 1854-56 survey stakes, but in an area of thirty townships only five corners were discovered which seemed to be authentic. These were widely scattered, and showed great errors. Between the 3d and 4th parallels the actual distance was found to be 25¼ miles, or an error of 1¼ miles in a 24-mile north and south line. East and west the error was approximately 2 miles in 30.

Throughout the territory, Sections 16 and 36, the school sections, had been given to the State of California by the United States Government for the benefit of the State school fund, the remainder of the land belonged to the United States, and this dual ownership increased the difficulty of making any adjustment. In June, 1902, the president of the colonization company and the chief engineer of the C. D. Co. went to Washington, explained the situation, and, on the advice of the General Land Office, an Act was prepared and passed in July, 1902, authorizing a resurvey of twenty townships of the land in Imperial Valley. The outside lines of these townships were re-run in 1903 and are known locally as the "Henderson" survey. It was more than 6 years, however, before the interior lines in these townships were re-run and the work was completed and approved.

In the mean time, it was impossible for the Land Office to issue patents to the settlers, and thus men practically owning from 160 acres to two and three times that area of extraordinarily fertile land, with a selling value of from $60 to $100 per acre, could offer no security for a loan with which to make permanent improvements. The United States land laws are extremely strict and severe with reference to a settler borrowing money with which to make final proof. Under such circumstances, the interest rate was naturally from 10 to 12% per annum, while the interest on deferred payments for the water stock was only 6%, so that the C. D. Co. suffered severely. However, it was not until 1909, more than 3 years after the control of the company was taken over by the Southern Pacific interests, that any suits were entered to foreclose on the collateral notes and mortgages secured by the water stock.

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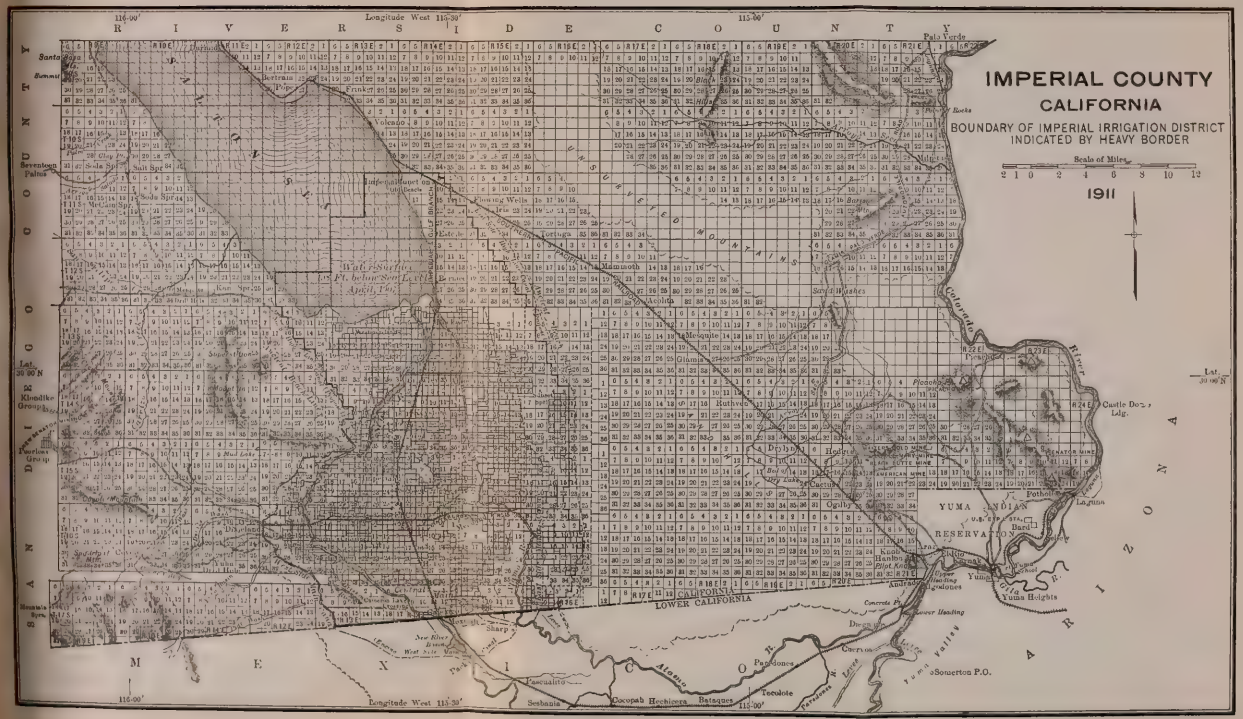

the lines to the north were retraced, but, some time later, when the survey had extended farther and the work of retracing the lines east of the Alamo River was commenced, it was discovered, by encountering natural features given by the notes of the 1856 survey, 2 miles or more out of correspondence, that there were serious errors. Exhaustive search was then made for the 1854-56 survey stakes, but in an area of thirty townships only five corners were discovered which seemed to be authentic. These were widely scattered, and showed great errors. Between the 3d and 4th parallels the actual distance was found to be 25¼ miles, or an error of 1¼ miles in a 24-mile north and south line. East and west the error was approximately 2 miles in 30.

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PLATE XLVII.
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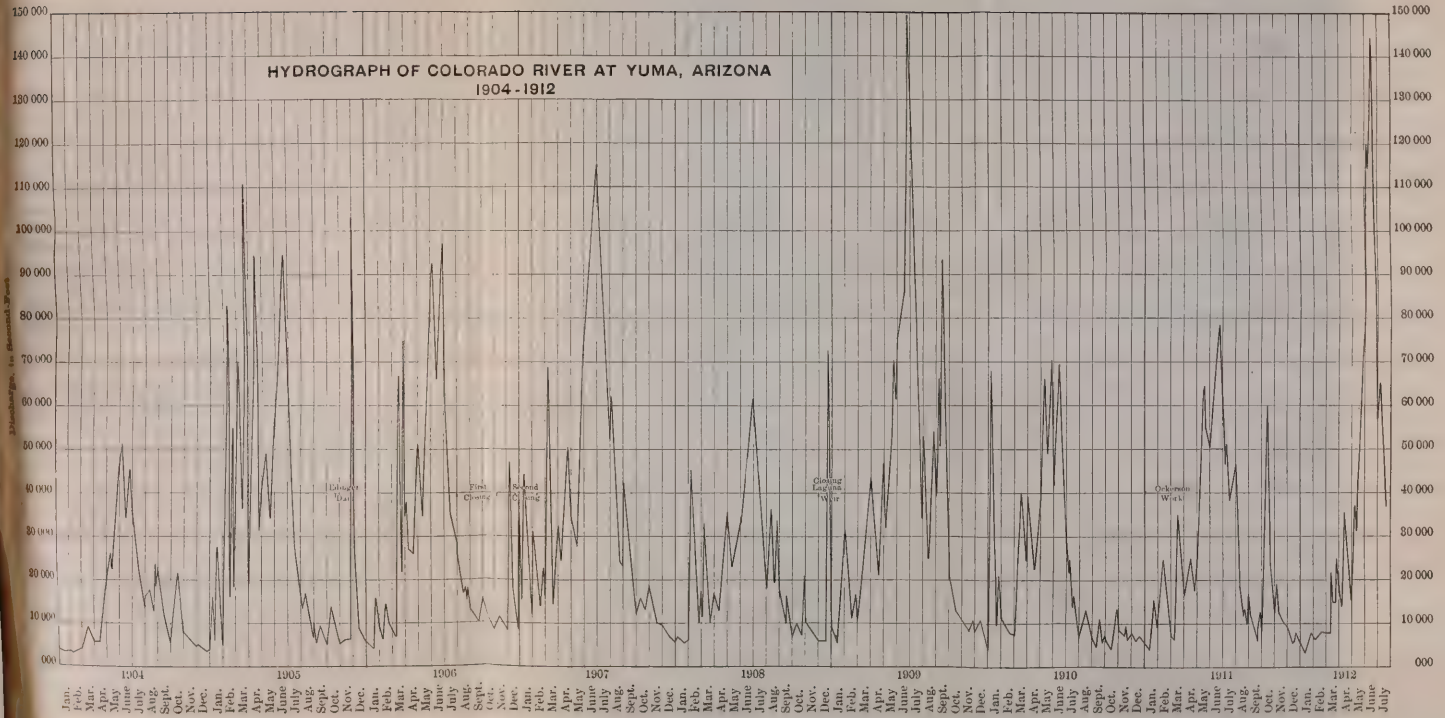


Soil Surveys of the Imperial Valley.—In the fall of 1901 the Bureau of Soils, United States Department of Agriculture, made a soil survey of Imperial Valley. On January 10th, 1902, a preliminary report, "Circular No. 9," was issued covering the 169 sq. miles of territory which had been examined.* The report doubtless presented the only possible conclusions, according to the information at that time extant regarding alkaline soils of such depth as are found in the Imperial Valley. It was very unfavorable, however, and calculated to deter sensible people from settling in the region. For example, one statement was as follows:

"One hundred and twenty-five thousand acres of land have already been taken up by prospective settlers, many of whom talk of planting crops which it will be absolutely impossible to grow. They must early find that it is useless to attempt their growth. * * * No doubt the best thing to do is to raise crops such as the sugar beet, sorghum, and date palm (if the climate will permit), that are suited to such alkaline conditions, and abandon as worthless the land which contains too much alkali to grow those crops."

The warning was reiterated in a subsequent report.† It seems certain that, had the territory not been already settled in very large measure when these reports were sent out, Imperial Valley would yet be unreclaimed.

Agitation in Favor of a Reclamation Service Project.—When the United States Reclamation Service Act was passed, in June, 1902, the crops produced in the Imperial Valley were causing a return of confidence in the region, and the extraordinarily rapid development was being resumed. The irrigation possibilities on the Colorado River had already been examined by the United States Geological Survey, and in 1903 plans for the Yuma Project were outlined. The engineers of the Service were convinced that no diversion from the Colorado for irrigation could be permanently successful where provisions were not made for preventing the heavy silt from entering the canals—that it would take an impractically large amount of dredging to keep canals leading directly from the river open for reasonably satisfactory delivery of water. The cost of the Laguna Weir, borne by the land owners of Imperial Valley alone, constitutes a serious burden, but, if

* "Field Operations of the Bureau of Soils, U. S. Department of Agriculture, 1901, p. 587."

† "Soil Survey of the Imperial Area, California (Extending the Survey of 1901), Advance Sheet of Field Operations of the Bureau of Soils, 1903."

borne by all the irrigable land in both valleys, the cost per acre would be reduced to approximately one-fourth. Mr. William E. Smythe, of San Diego, who has been very prominent in the work of the National Irrigation Congress, and has written extensively on irrigation generally, urged the people of Imperial Valley to join with the Yuma Project; that the enterprise would then be backed by the Government with unlimited funds; that they would be required to pay to the Government only a small portion of the money they were obliged in one way and another to pay the C. D. Co., and that they would eventually attain the laudable desire of owning and operating their own system. The Imperial Water Users' Association was accordingly formed, with Mr. W. F. Holt, of Redlands, Cal., as its President, and negotiations were at once instituted with the C. D. Co. to acquire its canal system. Mr. A. H. Heber, President of the C. D. Co., who acted for it in the matter, knew that the estimates of the Reclamation Service for the canal line into Imperial Valley, lying entirely on American soil, were at least $10 000 000, on account of the sand hills. He believed that the Alamo River for 40 miles was a very satisfactory main canal, and that by owning the 100 000-acre tract of the Mexican Co., building another waterway through Mexican territory would require the consent of the Mexican Co.; consequently, his idea regarding the values of the property were excessively high.

As a natural feature of these negotiations, and with a view to tempering such ideas as to price, the right of the C. D. Co. to take water from the Colorado was challenged. The navigability of that stream suddenly assumed serious commercial, national, and international importance. As usual in such cases, these questionings were carried to unfortunate extremes.

In the course of events, at a mass meeting held in Imperial on July 30th, 1904, Mr. Heber offered to have the price fixed by arbitration, one man to be appointed by him, one by the Imperial Water Users' Association, and a third to be selected by these two. This was not done, but the Engineers of the Reclamation Service made an appraisal of the C. D. Co.'s plant. The Imperial Water Users' Association appointed a committee headed by its president, Mr. Holt, to represent it in all negotiations, and it agreed with Mr. Heber on a price of $3 000 000. Mr. C. D. Walcott, Director of the U. S. Geological Survey, and of the Reclamation Service, made a report to the Secretary of the

Interior in the matter on September 15th and again on October 1st, 1904, that

"from present knowledge of the conditions, a recommendation to pay three million dollars for the property and rights involved is not justified."

On being advised of this report, an endeavor was ostensibly made by the Committee and Mr. Heber to secure different action. It appears, however, that, without intimating to Mr. Heber that it had changed front, the Committee agreed with the Reclamation Service authorities against buying the property on the agreed basis. With such unpardonably bad faith on the part of the Committee, it is not surprising that the matter ended with relations between Mr. Heber and the Reclamation Service so strained that further negotiations were impossible.

In a further report made by Director Walcott on January 4th, 1905, to the Secretary of the Interior, it was advised that, even if there were no complications because of the unfavorable soil report by the Department of Agriculture and

"the present desperate situation of many of the inhabitants of the Imperial Valley, it was a matter of grave doubt whether it was wise to become involved in the situation.

"Much of the land filed on is unsuitable for cultivation owing to the large amount of alkali and other adverse conditions of the soil. The water supply is deficient, owing to poor construction and accidents to the canal system, and although during the fall months there has been ample water, it appears that the people will not utilize it, and from best information are not planting crops to any considerable extent. Difficulties of handling the silt are very great, and if the Government is to take up the project, it must make enormous expenditures at once to prevent the country lapsing into a desert condition."

Shortly after this, it was announced by the Secretary of the Interior that the Assistant Attorney General had concluded that no law existed whereby it could deal with the problem of carrying water through Mexico.

The effect of the entire incident was to render the people of the valley antagonistic to the company, and at the same time split them into several factions. More important, however, was the effect of the severe criticisms of the plant and water rights of the C. D. Co., which had been given wide publicity. The company's credit, which had slowly but steadily improved since 1902, was again destroyed in South-

ern California and in the larger financial markets of the United States. Consequently, early in 1905, when these negotiations ended, the company was almost on the rocks.

Water Rights Attacked.—Because of the attacks on the right to take water from the Colorado, then well under way, a bill was introduced into the House of Representatives in January, 1904, at the request of the C. D. Co., declaring:

"That the water of the Colorado River for the irrigation of the arid land that may be irrigated therefrom is hereby declared to be of greater public use and benefit than for navigation, and the diversion of the water from said river, heretofore made and that which may in future be made, for irrigation purposes, in accordance with the laws of the respective States and Territories in which such diversion has been or may be made, is hereby legalized and made lawful.

"Section 2. That any person, firm, or corporation be, and is hereby, authorized to divert, take, and appropriate water from the Colorado River for the purpose of irrigation, in such quantity, subject to and under the State appropriation of the State of California, as now in force under the laws of said State." (H. R. 13 627, 58th Congress, 2d Session.)

The U. S. Reclamation Service had filed on some of the flood-waters of the Colorado in order to fill four large reservoirs between The Needles and Yuma, then under contemplation, and such filings were practically second only to those of the C. D. Co., so that the effect of this proposed legislation, other than on the C. D. Co., was null. The bill was bitterly opposed by Mr. Smythe, as representing the majority of the settlers in Imperial Valley. No attempt was made to amend the bill with a view of protecting all interests in a fair and equitable manner, but instead, under date of April 8th, 1904, the Acting Attorney General, Mr. Hoyt, in an opinion addressed to the Committee on Irrigation on Arid Lands, to which the bill had been referred, said:

"In view of these provisions [from the Treaty of Guadalupe-Hidalgo, February 2d, 1848; of the Gadsden Purchase, December 30th, 1853; and the Boundary Treaty of November 12th, 1884, between Mexico and the United States] and of the important irrigation projects now and hereafter to be carried on by the United States Government. I seriously doubt the wisdom of a surrender by Congress at this time of all control over the waters of the Colorado River."

Accordingly, the Committee reported* requesting the Secretary of the Interior to investigate and report to Congress on the various questions involved in the use of the waters of the Lower Colorado River, with a view to determining their availability for irrigation, and recommend any legislation which might be necessary. Under this resolution, Director Walcott reported through the Secretary of the Interior, on December 24th, 1904, that governmental control of the waters of the Colorado River was absolutely necessary to obtain the best results; that, the river being navigable, water appropriations under the Arizona and California laws were not valid; suggesting that legislation be along the line guarding present navigation interests; confining diversions heretofore made to the extent of actual beneficial use of the waters diverted upon land capable of producing remunerative crops, and making the same appurtenant to specific tracts of land; and making the approval of the Secretary of the Interior necessary with respect to future appropriations.

Mexican Concession Secured.—Failing to secure an adjustment of water rights at the hands of Congress, Mr. Heber went at once to Mexico and quickly obtained a concession from President Diaz, which was ratified by the Mexican Congress on June 7th, 1904.

This concession authorized the Mexican Co. to carry, through its canal system in Mexico, 284 cu. m. per sec. (approximately 10 000 sec-ft.), to be diverted from the Colorado River in United States territory by the C. D. Co. and turned over to the Mexican Co. at the boundary line; to construct an intake on Mexican territory, and connecting with the said canal system, and divert through such intake 284 cu. m. per sec., to be used in the irrigation of lands in Mexico and in the United States, but with the proviso, "without injuring the rights of any third party nor the navigation as long as the river is destined for navigation"; that, of the water carried in the canal, enough should be used to irrigate the lands in Mexico susceptible of irrigation by gravity to an amount not exceeding one-half the total volume; that the Mexican Co. should begin surveys within 6 months, and within 12 months file, with the Secretary of Development, maps in duplicate of the proposed extensions and betterments, together with a descriptive report, and entirely complete the same within 7 years;

* House Joint Resolution Nos. 147, 133. State. 591.

that the company should pay into the Inspection Fund, as is customary in all concessions granted by the Mexican Government, a sum, in this case $300 (Mexican money), per month, and should be subject to inspection by an engineer appointed by the Secretary of Development; granting the company the right of eminent domain over private property and defining the process by which condemnation could be carried out—incidentally with minimum possible difficulty—and permitting importation once for all, free of customs or duty, all equipment and apparatus necessary for the construction of the proposed extensions and betterments, together with freedom from all taxes, except stamp tax, for a period of 10 years; stipulating that under no circumstances should the company sell or mortgage the concession to any government or foreign state, nor admit it in partnership; that the company should be subject to the laws and rulings now in force, and which in future may be enacted, for the supervision and use of waters; particularly specifying that the company and its assigns shall always be considered as Mexican corporations, though all or any of its stockholders should be foreigners; that the corporation should be subject to the jurisdiction of the Courts of the Republic in all affairs emanating within Mexican territory, and that such stockholders should never be able to allege the rights of foreigners under any circumstances, but have the rights and the methods of establishing the same as the laws of the Republic grant to Mexican citizens, so that, in any matters, diplomatic or foreign agents should not have any interference.

Condition of Plant in the Summer of 1904.

From the first, there was a great deal of trouble with the Chaffey Head-gate, chiefly because its floor was not down to the bottom grade line of the canal, as originally planned. As has been explained, this gate was a temporary structure, but well and substantially built. Just as it was being covered up by the operations of the dredge, *Alpha*, cutting the main canal into the permanent concrete head-gate from below, in 1906, the writer examined it carefully and found it in an excellent state of preservation. The floor was so high, however, that it was necessary, during the low-water seasons of 1902-03 and 1903-04, to cut a by-pass around the gate, and close it on the approach of the summer floods. When the Mexican concession was obtained, the first Mexican intake was cut from the river to the main canal, as shown in Fig. 13.

In the winter of 1902-03 there had been shortages of water in the valley, due to the fact that the main canal had not been completed to its final depth; and, with the apparatus and available funds on hand, it was impossible to keep the water supply up to the demands when the river fell exceedingly low. In the winter and early spring of 1904, another water shortage caused considerable damage in the valley, and claims amounting to $500 000 were presented to the company. Every one of these was settled out of Court, however, in 1905 and the early part of 1906, with a payment of less than $35 000, taken entirely in water and water stock, and the writer believes that every claim was fairly settled, at least as far as the settlers were concerned.

Below the intake the first 4 miles of the Main Canal caused much worry, due to the extent to which it silted up during floods, but, with this exception, the plant of the C. D. Co. was in quite satisfactory condition. The canals were generally well located and in fair condition, and the structures, though of redwood and not concrete, were substantially built according to good design, and were in excellent condition. The canals in the distribution systems of the mutual water companies were silting up constantly, on account of the muddy water. In part, this was unavoidable, but was largely due to uneconomical methods of water deliveries when dealing with muddy water, particularly in serving any settler on his demand, regardless of the very low velocity, if no one else wanted water from the lateral during the same time. The silt problem in the distribution systems of these companies, however, is as simple as it will ever be for any lands irrigated along the Lower Colorado. The financial status of the various mutual companies was quite good, and they had generally established a small but satisfactory credit with the local banks.

To avoid excessive silt depositions in the first 4 miles of the canal, in February and March, 1904, the Best Waste-gate, so-called, was put in 8 miles below the intake, where water could be wasted from the Alamo channel through the Quail River into the Paredones River and thence into Volcano Lake. This was a wooden **A**-frame, flash-board gate, 60 ft. long, but it was carried away in June, 1906, by the side-cutting of the banks while the Alamo channel was being enlarged by that year's summer flood. The idea was to divert a large quantity of water during the flood season of 1904, waste it through the Best Gate, and in this way scour out the upper portion of the canal. At

first the action was as expected, and some 2 ft. in the bottom were carried away. When the river reached its maximum height during the summer flood of 1904, however, and carried an excessive silt content, particularly of the heavier and sandy type, this scouring action was entirely overcome, and the bottom of this stretch was raised approximately 1 ft. higher than during the previous year.

The Silt Problem.—This action accentuated, and properly impressed on the engineers of the C. D. Co., the seriousness of the silt problem in diverting the Colorado River water. Generally speaking, during flood stages, the water carries all the silt it can transport, and the faster the current the larger the particles it picks up and carries along. It is certainly desirable, and probably essential, to provide settling basins at or immediately below the diversion point, in which water can be practically stilled and thus insure the deposition of the heavier silt having very slight fertilizing value, and the admission of only such partly clarified water into the canals. Unless some provision is made, as at the Laguna Weir, the diversion canal immediately below the head-gate must act as a settling basin, which is just what happened from the very beginning in the canals of the C. D. Co.

The results of such excessive silting were obviated in various ways, largely by the construction of new intakes, until the diversion of the entire river occurred, and the permanent head-gate was put in service in 1907. The clam-shell dredge, *Delta,* was utilized intermittently to remove the deposits until 1910; then a submerged weir was built across the river, to raise the water at the intake; and lastly large suction dredges were operated just below and just above the regulating gates.

Rather carefully kept records indicated that the bed of the canal at the Lower Heading was raised a little more than 5 ft. between March 1st, 1907, and March 1st, 1910, most of this taking place in the first 6 months. The bed of the Alamo near Sharp's Heading was raised approximately 2 ft. in the same time, and there is constant deterioration all along between these points on the Alamo channel. The reduction of capacity in the larger canals has been noteworthy, but the maximum effect is shown in the smaller laterals constituting the distribution systems of the various mutual water companies.

Mr. Robert G. Kennedy states* that on the Bari Doab Canal from the Punjab River, the canals in Sind from the Indus and Shwebo,

*"The Prevention of Silting in Irrigation Canals," *Minutes of Proceedings*, Inst. C. E., Vol. CXIX, 1895, pp. 281–290.

and the Mandalay canals in Burmah, it appears that in a non-silting and non-scouring channel the mean velocity is independent of the width, but increases with the depth of the channel, according to the equation:

$$V_0 = 0.84\, d^{0.64},$$

in which V_0 = the mean velocity of a non-eroding, non-depositing current; and d = the depth for fine sand-silt, the constants varying slightly with the kind of silt.

He also points out the exceedingly important deduction that during flood stages in the river, the diversion of large quantities of water in an effort to scour away silt depositions in the upper reaches of canals will have the opposite result, because of the excessive silt contents of the water diverted.

The same rule probably applies fully in the case of canals carrying Colorado River water when they are free of vegetation. In point of fact, however, rank growths of tules and willows spring up, on the banks and berms and along the edges out into the water, with such rapidity as to increase tremendously the deterioration of carrying capacity, particularly in the smaller canals. Furthermore, the rate of deterioration in these laterals increases with the decrease in channel efficiency. The maintenance of the district distribution systems, therefore, consists, in large part, in keeping down and removing the brush and tules.

The various distribution systems were ordinarily designed and built on the basis of a capacity of 1 sec-ft. per 120 acres of land thereunder, although in some cases the ratio was decreased to 1 sec-ft. per 93 acres (8 in. vertical depth of water in a month). It would have been just as well, indeed considerably better, and of course cheaper, to have made the canals much smaller, for they were put into service when only a small percentage of the land was in cultivation, and, as they carried only a fraction of their capacity, they very soon silted up badly. Removing the silt deposition and the accompanying tule growths is fully as expensive as the excavation of the original section.

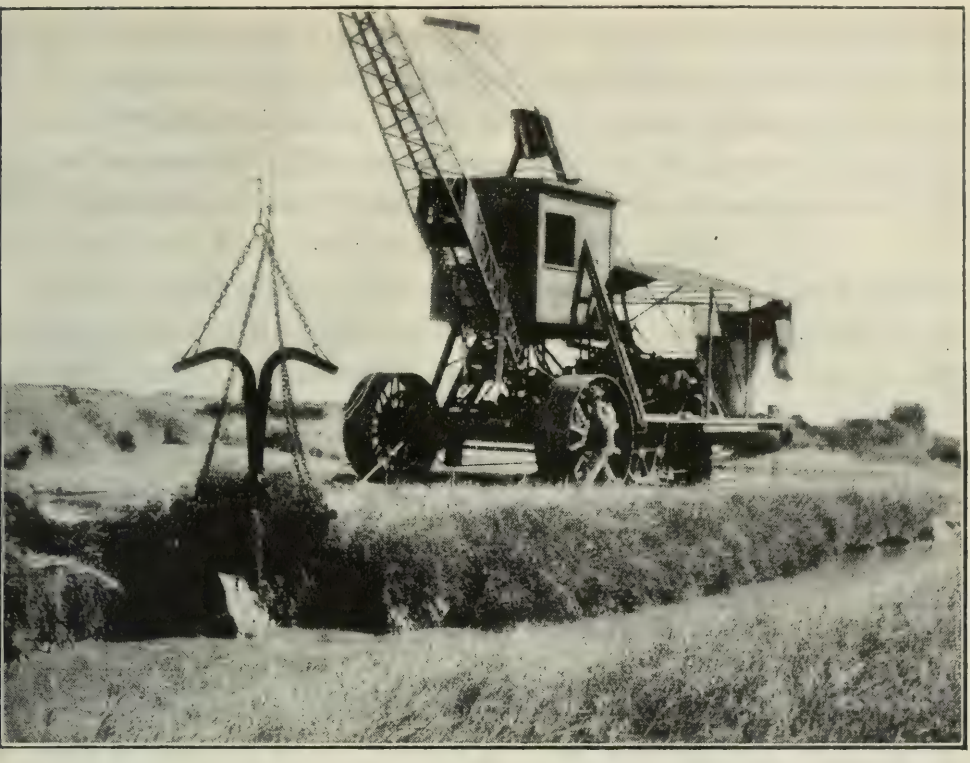
This needlessly large excavation was required by the contract provisions under which the C. D. Co. built the distribution systems of the various mutual water companies, and such provisions at the time were necessary to assure colonists that the water supply would be ample. In the construction of the first lateral canals built, however,

the leaving of inside berms was a defect which should have been avoided. These flat stretches, usually kept damp and seldom deeply submerged, afford ideal conditions for tule growths, and should be studiously avoided in this region.

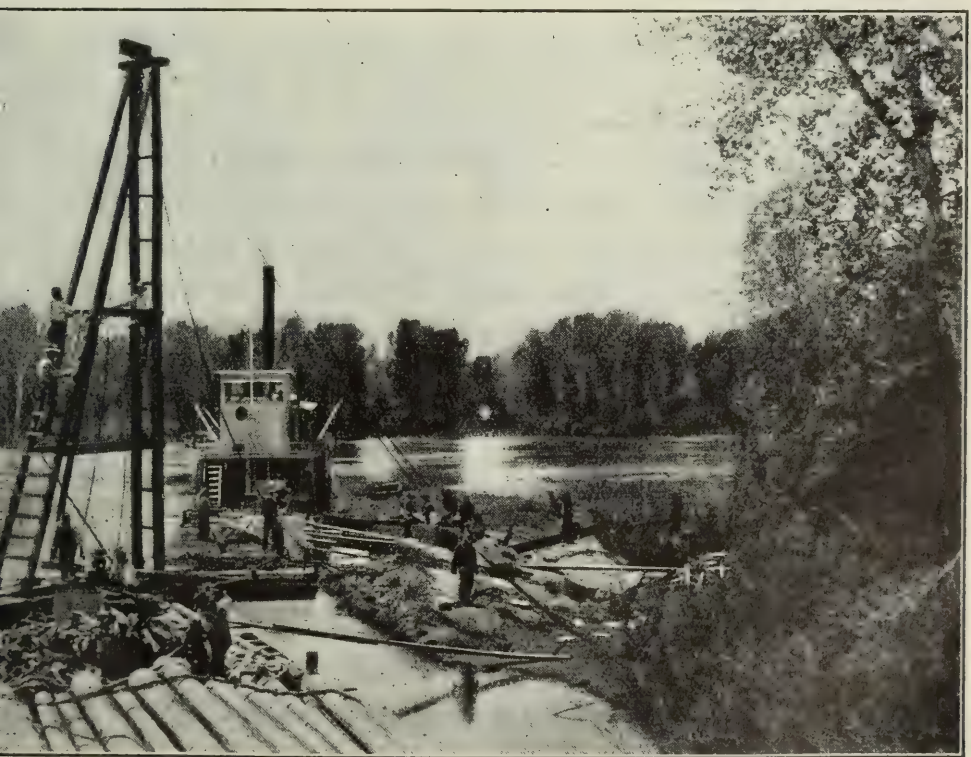
Canal Maintenance.—In general, the best method of clearing away the brush, tules, and deposited silt in the smallest canals has been found to be by Mexican or Indian hand labor. The presence of checks and other canal structures at relatively close intervals makes the use of machinery of questionable economy. For the large canals, "V's", dragged by horses or traction engines, portable floating dipper-dredges, Lidgerwood cross-drags, portable clam-shell dredges, and a number of devices designed by local inventive geniuses have been tried with varying success. The results in all cases depend so greatly on the efficiency with which they are handled and the local conditions under which they work that it will not be profitable to attempt to give any cost figures—indeed, with the exception of Imperial Water Company No. 1, no cost-keeping worthy of the name has been attempted.

Perhaps the most satisfactory appliance for cleaning canals too small for floating dredges is a clam-shell bucket arranged on wheels so that it may follow along the bank. (Fig. 17.) The C. D. Co. has two of these machines, manufactured by the Stockton Iron Works, Stockton, Cal., which cost $5 000 each, f. o. b. factory. These consist of a clam-shell bucket having a capacity of 15 cu. ft., with a 40-ft. steel boom carried on an all-steel frame. The maximum width is 14 ft. The power is supplied by a 15-h.p. Atlas gasoline engine, manufactured in San Francisco, and the machine is self-propelling, with two speeds forward and one reverse. No definite figures, including deterioration and cost of moving from one job to another, are available, but it is understood that the cost of handling material with these machines is about 13 cents per cu. yd.

For handling silt in the upper reaches of the Main Canal along the river, the large 4-yd. dipper dredge, *Alpha,* used in the original construction, was perhaps the most efficient of all agencies for the first few years until the waste banks along the canal became too high to permit of its further use; it handled material for about 6 cents per cu. yd. A suction dredge, the *Beta,* equipped with a 12-in. Kroh centrifugal pump—manufactured in San Francisco—was tried very soon after the canals were put into service, but too much difficulty was



IG. 17.—STOCKTON CLAM-SHELL DREDGE CLEANING CANALS IN IMPERIAL VALLEY.



IG. 18.—THIRD ATTEMPT TO CLOSE BREAK AT LOWER MEXICAN INTAKE, JUNE 1ST, 1905.

aused by roots clogging the pipes and machinery. Mr. H. W. Blaisdell, f Los Angeles, one of the principal stockholders of the C. D. Co., evised a rotary cutter for use at the end of the suction pipe, but it as not successful. This dredge was used at the Lower Heading, in he construction of the Rockwood Gate, and in the subsequent diver- ion work until June, 1907, when it was dismantled.

In the central main in the valley, and also in the Alamo channel ust above Sharp's Heading, a 2-cu. yd. dipper dredge, the *Gamma,* has een used almost continuously since it was put in operation in 1904, emoving material at about 5 cents per cu. yd.

The clam-shell dredge, *Delta,* described in some detail later, has one excellent service in silt removal and incidental levee building, as ell as in channel straightening, since its arrival on the work in ovember, 1906. It is now engaged in building cut-offs and making eneral channel improvements, rather than removing the silt deposits irect.

In the summer of 1910 an arrangement was entered into between he various mutual water companies combined and the Receiver of he C. D. Co. whereby the former were to furnish the money and build suction dredge and rent it to the latter for 10% annually on its first ost. This dredge was built just below the concrete head-gate, and its peration is confined to the American side of the line, the contract eing entered into with the North American Dredging Company, of os Angeles, on December 10th, 1910, for the construction and equip- ent of an exact duplicate of one of the latter company's dredges in an Pedro Harbor, for $57 300. After being put into service it was ound necessary to remodel the upper deck, in order to make the uarters of the crew suitable for the climatic conditions, at a cost of 50, and a bonus of $2 200 was paid for completion 11 days ahead the contract time—4 months—making the total cost $60 450, exclu- ve of engineering, inspection, and legal expenses, which brought the and total cost up to approximately $63 600. This dredge, the *mperial,* has a hull 105 ft. long, 55 ft. wide, and 8 ft. deep, and is quipped with a 15 by 60-in. Kroh centrifugal pump driven by a rtical compound engine, steam being supplied by a 250-h.p. marine- pe boiler. This dredge handles the silt deposits in the enlarged ction of the canal below the head-gate at the rate of about 200 cu. yd. r hour lifted to an average height of 35 ft., at a cost of from 5 to 7

cents per cu. yd., exclusive of interest, taxes, and depreciation, using crude oil fuel at $1.40 per bbl., equivalent to coal at $5.60 per ton.

The *Imperial* was equipped with a cutter for stirring up the material, but this was found to be unnecessary for handling the silt deposits just below the head-gate, and the cutter engine, of vertical compound type, with 8 by 15 by 10-in. cylinder, was installed on the barge, *Silas J. Lewis,* mentioned later, in the canal above the head-gate to run the 10-in. Kroh pump formerly on the *Beta,* the resulting dredge being known as the *El Centro.* Under like conditions, the cost of handling material with the *El Centro* is approximately the same as with the *Imperial.*

With these two suction dredges, it is claimed that the bed of the Main Canal has been lowered approximately 5 ft. above and at the head-gate and for a distance of 3½ miles below, diminishing gradually to nothing throughout the next 2½ miles. If future experience confirms such results, it would seem that the periodic dredging of silt depositions from a settling basin near the intake, at a cost of from $30 000 to $40 000 per annum, will solve the silt problem in the Imperial Valley canal system, except for the very fine silt which cannot be gotten rid of except by allowing the water to be quiescent for some time.

The following general cost figures on maintaining the 373.25 miles of canals of the distribution system of Imperial Water Company No. 1 during 1911 are taken from the annual report of the Superintendent, R. S. Carberry, Assoc. M. Am. Soc. C. E.

Cleaning 465 miles of canal cost $60.64 per mile. The figures in 1910 are 562 miles at $43.81 per mile, and the average cost for the last 6 months of 1909 was $73.16. Clearing on 194 miles of canal cost $35.39 per mile. Cutting brush on 392 miles cost $20.71 per mile. The figures in 1910 were 346 miles at $43.47 per mile, and $60.65 per mile for the last 6 months of 1909.

In this report it is stated that canal "V'ing" is the best method for cleaning canals, generally speaking, and the company owns three "V's", each costing about $600, and three caterpillar traction engines to operate them, each costing $4 200. During the year, 363.8 miles of canal were "V'd" at $58.81 per mile, as compared with 362 miles in 1910 at $60.74 per mile, the details being as follows:

"V'ing"	$16.76
Repairs to engines	16.29
Repairs to "V's"	5.00
Fuel and oil	5.96
Mexican labor following "V's"	14.80
Total average cost per mile	$58.81

During the year, 1 415 miles of canal were worked on, so that the whole system was covered in various ways nearly four times during the year. A small portion of the system was not worked on at all, so that this statement gives some idea of the difficulty in maintaining the system.

The cost of building 117 new structures was $6 278.75, and the cost of repairing old structures was $4 145.05, making the total cost of structure maintenance and renewal $10 423.80. The average number of men employed per day (300 working days per year) was 162, or 0.43 man per mile of canal per day, in addition to teams and machinery. The bottom width of the canals constituting this system varies from 20 to 5 ft.

Canal Operation.—The mutual water companies have never considered delivering water to stockholders in rotation, but instead, without exception, supply any water user on demand, even though he may be at the very end of a long lateral and the only person desiring water from that lateral at the time. Thus, naturally, exceedingly small quantities of water are carried occasionally in every canal except the very largest laterals, and the result is low velocities and heavy silt deposition and canal deterioration. The feeling seems to be general that the additional cost of maintaining the various distribution systems is more than offset by the advantages or convenience of the water users in obtaining irrigation water at all times on 24 hours' notice. The amount which the maintenance cost of canals could be cut by adopting a rotation system of delivery is problematical, but must be between 35 and 65 per cent. This fact should be borne in mind in making comparisons with the cost data just given.

The Fourth or Lower Intake.

This is such a very important matter that the reasons for digging the lower Mexican intake and the method of handling it when

completed are given by quoting from Mr. Rockwood,* the man who did it.

"As soon as the summer flood (1904) dropped and I discovered that instead of the bottom being lower it was approximately 1 ft. above that of the year previous, we adopted the only means at our command to attempt to deepen the channel.

"Knowing the character of the material to be removed, we knew that with the dredging tools which we had (4-yd. dipper dredge *Alpha* and 12-in. suction dredge *Beta*), it would be impossible to dredge out this 4 miles of canal in sufficient time for the uses of the valley, providing the water in the river should drop as low as it had the previous year. The dredges were brought back, however, and put at work, but the result proved, as I had anticipated, that it would take practically all winter to dredge the canals; that is, it would take all winter to provide new machinery, even if we had the money; and in hopes, then, that it might possibly prove effective, I employed the steamer *Cochan*, and, placing a heavy drag behind it, ran it up and down the canal in hopes that by stirring up the bottom there would be sufficient velocity in the canal itself to move the silt deposits on below the 4-mile stretch to a point where I knew the water had sufficient velocity to keep the silt moving. A month's work, however, with the steamer proved that the work being done by it was inadequate.

"*The Great Problem.*—We were confronted then with the proposition of doing one of two things, either cutting a new heading from the canal to the river below the silted 4-mile section of the canal, or else allowing the valley to pass through another winter with an insufficient water supply. The latter proposition we could not face for the reason that the people of the Imperial Valley had an absolute right to demand that water should be furnished them, and it was questionable in our minds as to whether we would be able to keep out of bankruptcy if we were to be confronted by another period of shortage in this coming season of 1904-1905.

"The cutting of the lower intake, after mature deliberation, and upon the insistence of several of the leading men of the valley, was decided upon. We hesitated about making this cut, not so much because we believed we were incurring danger of the river's breaking through as from the fact that we had been unable to obtain the consent of the Government of Mexico to make it, and we believed that we were jeopardizing our Mexican rights should the cut be made without the consent of the Government. On a telegraphic communication, however, from our attorney in the City of Mexico, to go ahead and make the cut, we did so under the presumption that he had obtained the necessary

* "Born of the Desert—Imperial Valley In Its Making Not A Dream—A Brief History of the California Development Company." By C. R. Rockwood. Second Annual Magazine Edition. *Calexico Chronicle*, Calexico, Cal., May, 1909.

permit from the Mexican authorities. It was some time after this, in fact after the cut was made to the river, before we discovered that he had been unable to obtain the formal permit, but had simply obtained the promise of certain officials that we would not be interfered with, providing that plans were at once submitted for the necessary controlling structures to be placed in this heading.

"Reasons Why.—This lower intake was constructed, not, as is generally supposed, because there was a greater grade from the river through to the Main Canal at this point. The grade through the cut and the grade of the Main Canal above the cut were approximately the same, but the cut was made at this point for the reason that the Main Canal below the point where the lower intake joined it, was approximately 4 ft. deeper than the Main Canal through the 4 miles above this junction to the Chaffey gate, consequently giving us greater water capacity. In cutting from the Main Canal to the river at this point, we had to dredge a distance of 3 300 ft. only, through easy material to remove, while an attempt to dredge out the Main Canal above would have meant the dredging of 4 miles of very difficult material. We began the cut the latter end of September and completed it in about 3 weeks.

"As soon as the cut was decided upon, elaborate plans for a controlling gate were immediately started and, when completed early in November, were immediately forwarded to the City of Mexico for the approval of the engineers of the Mexican Government, without whose approval we had no authority or right to construct the gate. Notwithstanding the insistence of our attorney in the City of Mexico and various telegraphic communications insisting upon this approval being hurried, we were unable to obtain it until 12 months afterward, namely, the month of December, 1905.

"Unprecedented River Conditions.—In the meantime serious trouble had begun. We have since been accused of gross negligence and criminal carelessness in making this cut, but I doubt as to whether any one should be accused of negligence or carelessness in failing to foresee that which had never happened before. We had before us, at the time, the history of the river as shown by the daily rod readings kept at Yuma for a period of twenty-seven years. In the twenty-seven years there had been but three winter floods. In no year of the twenty-seven had there been two winter floods. It was not probable, then, in the winter of 1905, that there would be any winter flood to enlarge the cut made by us, and without doubt, as it seemed to us, we would be able to close the cut before the approach of the summer flood by the same means that we had used in closing the cut for three successive years around the Chaffey gate at the head of the canal.

"During this year of 1905, however, we had more than one winter flood. The first heavy flood came, I believe, about the first of February,

but did not enlarge the lower intake. On the contrary, it caused such a silt deposit in the lower intake that I found it necessary, after the flood had passed, to put the dredge through in order to deepen the channel sufficiently to allow enough water to come into the valley for the use of the people.

"This was followed shortly by another heavy flood that did not erode the banks of the intake, but, on the contrary, the same as the first, caused a deposit of silt and a necessary dredging. We were not alarmed by these floods, as it was still very early in the season. No damage had been done by them, and we still believed that there would be no difficulty whatever in closing the intake before the approach of the summer flood, which was the only one we feared. However, the first two floods were followed by a third, coming some time in March, and this was sufficient notice to us that we were up against a very unusual season, something unknown in the history of the river as far back as we were able to reach; and, as it was now approaching the season of the year when we might reasonably expect the river surface to remain at an elevation that would allow sufficient water for the uses of the valley to be gotten through the upper intake, we decided to close the lower.

"Five Floods in One Season.—Work was immediately begun upon a dam similar to the ones heretofore successfully used in closing the cut around the Chaffey gate. The dam was very nearly completed, when a fourth flood coming down the river swept it out. Work was immediately begun on another dam which was swept away by the fifth flood coming down during this winter season."

These closings of the by-passes or cuts around the Chaffey Gate were effected by throwing a barrier of brush across the cut and dragging earth over it with Fresno scrapers, pushing it into the water on the up-stream side, thus gradually rendering the barrier impermeable and then building it up as an earthen dam. In attempting to make the closure here mentioned, in March, a small pile-driver was rigged up on the end of the *Alpha,* and one line of 8 by 8-in. pine timbers, 3 ft. apart, was driven across the opening about 3 000 ft. west of the river bank, and an 8 by 8-in. waling was bolted to each pile above the water surface. Brush fascines were then made up, and all the sand bags available—about 10 000—were filled in readiness. Simultaneously from each side, brush fascines with the brush ends up stream, were piled above the piling and weighted down with sand bags, making alternate layers of fascines and bags, until the water was confined to a 30-ft. channel in the center. This barrier was about 20 ft. thick up and down stream. The opening was then spanned with long cottonwood timbers and a similar brush-sand-bag construction

was built upon them. The supporting timbers were then shattered with dynamite, letting the mass drop into the opening. At the same time a large quantity of brush was thrown in above and allowed to float into the opening to help close it. In this way the barrier across the opening was built above water and teams passed over it dragging in dirt from both sides, the flow being reduced so greatly that the dredge below it nearly went aground. With a few thousand more sacks of earth to place along the upper toe of the barrier, the work would have been successful. As it was, the structure was undermined, settled down, and eventually failed entirely.

In this attempt 10 000 sacks were used, 8 days' time with the dredge at $100 per day, and 225 men-days time of Indian labor at $1.50 per day. This makes the total cost of closing about $1 800.

Instructions were then given to move the dredge up close to the river bank, where the soil was thought to be better, and make another attempt. The current through the break, however, was too swift, and instructions were given to go up the old Main Canal to the upper Mexican intake to stop it, which was done, using the method which had failed below.

A similar method was used to throw the water through the Alamo Waste-gate on its completion in June, 1905, 3 months later, 30 000 sacks of earth being filled in readiness and every one used. This barrier dam was thrown across the channel carrying 2 500 sec-ft. of water and with a total or final head of 10 ft. This has always seemed to the writer to have been a most remarkable achievement, the only equipment at hand being a skid pile-driver and Fresno scraper teams.

To resume Mr. Rockwood's narrative:

"About this time, I left for the East and at the earnest solicitation of the Imperial Water Company No. 1, which agreed to advance $5 000 for the effort, a third* attempt to close the break was made under the direction of Mr. C. N. Perry and the superintendent of Imperial Water Company No. 1, Mr. Thomas Beach. On my return from the East, on the 17th of June, I found them heroically attempting to stop the break, with the waters so high in the Colorado that all of the banks and surrounding lands were flooded, and I immediately stopped the work as we realized fully that nothing could be done until after the summer flood had passed.

"The Colorado on a Rampage."—At this time, the lower intake had been enlarged from a width of about 60 ft., as originally cut with the

* Really, the second serious attempt, Fig. 14.

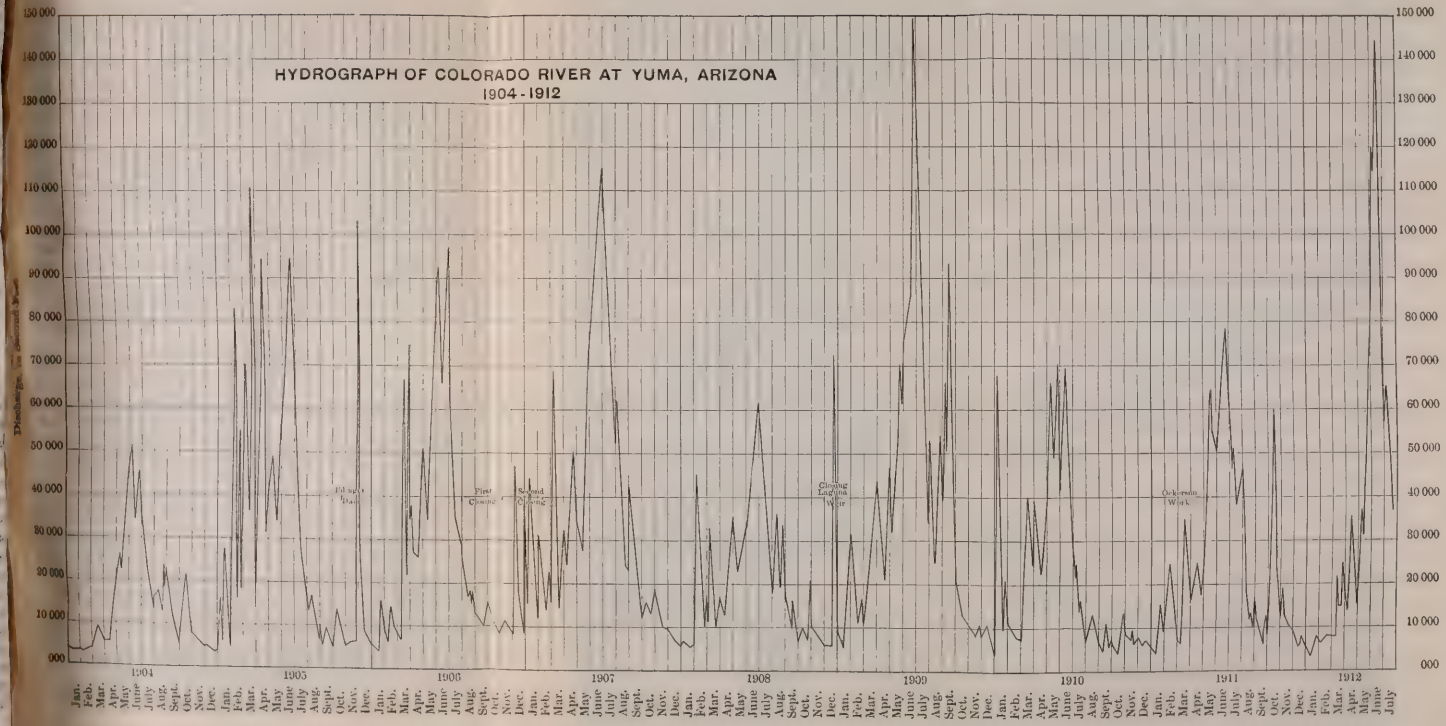
dredger, to a width of possibly 150 ft., and it did not then seem probable that the Colorado River would turn its entire flow through the cut, but as the waters of the river began to fall, the banks of the intake began to cave and run into the canal, the banks of the canal below the intake fell in and, as known by most of the residents of the valley, the entire river began running through the canal and into the Salton Sea in the month of August of this year of 1905."

Plate XLVII shows the discharge at Yuma to have been an unprecedented sequence of floods from the Gila water-shed. Indeed, the precipitation throughout all that region traversed by the Southern Pacific line from Yuma to very near El Paso during this period was quite without precedent. Track ballasted with local material, which had always proved satisfactory, was during this year the despair of the entire Maintenance of Way Department, and for months trains were allowed to go over it only at half speed and with lurchings of the coaches and Pullman sleepers like ships at sea.

Mr. Rockwood's statement gives a very fair presentation of the matter as he viewed it. The writer is perhaps as well aware as any one that the river was diverted through this cut into the Salton Sea, and when he first inspected the situation in August, 1905, he felt, like practically all other engineers who gave the matter cursory consideration, that making this cut was a blunder so serious as to be "practically criminal." After 4 years of more or less bitter experience with the region, he is perfectly convinced that, matters having gotten into such condition, making the cut was absolutely imperative and by all means should have been done. The difficulty had not been any tendency whatever to divert the entire river, but—very much to the contrary—to induce enough water to go that way. Up to that time, a head-gate to prevent too great a quantity of water from entering the canals was of far less importance than some means of maintaining their carrying capacity. That a head-gate should have been provided is, of course, self-evident. It would have been utter folly, however, to have put a flash-board gate of any type directly in the diverting channel, because of the drift which would have accumulated against it. Nothing less than a structure containing immense openings could have been used without insuring that the cut would be choked up. This type of construction was practically unused in western irrigation works at the time, and would have cost a great deal of money, therefore, considering the financial condition of the C. D. Co., it is plain that the only

1270 A.

PLATE XLVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1270.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.



practical thing would have been a gate, not in the cut itself, but in a by-pass, and built with the idea of closing the by-pass on the approach of the summer floods and using this gate as much as possible. It was not alone the straitened financial condition of the C. D. Co. and the situation generally in which it found itself which resulted in there being no permanent diversion works put in; two other important factors entered. The first was the practical change of management, from a construction point of view especially, to the Chaffeys in the summer of 1900; back to Mr. Rockwood in February, 1902, and internal difficulties in the C. D. Co. late in 1904. The second—indirectly connected with the first—was the hesitancy of the management to provide permanent head-works before the technical men in the corporation had agreed as to what the situation demanded. The real mistake was not in "putting all the eggs in one basket," but in not "then watching that basket." Obviously, no one could be responsible for doing such a thing without realizing the need for watching it most carefully and being fully prepared to take most aggressive action should occasion arise.

Southern Pacific Loan.

Early in January, 1905, it occurred to the management of the C. D. Co. that the phenomenal development of traffic furnished to the Southern Pacific Railroad by the Imperial Valley warranted the hope of financial assistance from that corporation. Mr. Julius Kruttschnitt, Director of Maintenance and Operation of the Harriman Lines, declined to consider the proposition, but Mr. Harriman, on being approached, was at once interested and ordered an investigation and report. As a final result, the Southern Pacific Company agreed to loan $200 000 on condition that 6 300 shares of the capital stock be placed in the hands of a trustee to be named by the Southern Pacific Company until the loan should be repaid, and taking over the management of the property until that time. Accepting a loan under such conditions was seriously objected to by a large part of the company's stockholders, but at the annual meeting in Jersey City, in May, a board was elected pledged to the arrangement. On June 20th, 1905, the Southern Pacific Company took over the management of the property, with Mr. Epes Randolph, President of the Associated Harriman Lines in Arizona and Mexico, as President, and

Mr. W. J. Doran, of Los Angeles, as the trustee mentioned in the contract. Both these gentlemen are still acting in these respective capacities. When the loan was arranged, and even when it was finally consummated, the railroad officials in San Francisco and the East did not consider the conditions along the river worthy of serious concern.

Mr. Rockwood was retained temporarily as Assistant General Manager and Chief Engineer, as members of the Southern Pacific management were entirely unfamiliar with the affairs of the C. D. Co.

Third Attempt to Close the Break.

As soon as the summer flood of 1905 began to recede, work was started. Immediately opposite the lower intake was an island, later dubbed Disaster Island, about $\frac{5}{8}$ mile long and $\frac{1}{4}$ mile wide, consisting really of a sand bar on which quite a growth of cottonwood and arrow weed had accumulated. A line of piling, 12 ft. from center to center, was driven from the upper end of this island to the Mexican shore, as shown in Fig. 15, and between this piling was woven barbed wire and brush. The theory behind this work was that, by spreading over a great width the water passing down the west channel and into the lower intake, a sand bar would be created, thus choking off the flow and gradually forcing all the water into the east channel. On July 15th about one-third of the river flow was going down the old channel and two-thirds toward the Salton Sea, and the result of this endeavor was still problematical. By August 1st a bar, approximately 2 800 ft. long, had been formed, but there was an opening, approximately 125 ft. long, through which the rush of water was too great to be controlled with the means at hand, and the work was abandoned. Up to this date, about $30 000 had been expended on the foregoing endeavors to close the break.

Various Suggestions for Handling the Situation.

At this time it was evident to all that the low-water flow of the Colorado would be entirely diverted into the old Alamo overflow channel and thence to the Salton Sea. The elevation of the water surface at the head of Disaster Island, with a flow in the river of 10 000 sec-ft., was approximately 100 ft., while the bottom of the Salton Sea is approximately — 287 ft., making the total fall in that direction 387 ft. The distance was about 95 miles by the watercourses, so that the average fall was 4.01 ft. per mile. Toward the Gulf the fall was 100 ft., and the distance to

ide-water was approximately 80 miles, or a fall of 1.25 ft. per mile. The continually diminishing quantity of silty water going down the old channel as the summer flood receded was constantly raising the bed along that direction, the action being rapid enough to be noticeable almost daily. In all probability there were about 6 months ahead during which the flow of the water would be low, and before this period should elapse the river must be re-diverted or the consequences would be most serious.

The plant and the salt deposits of the New Liverpool Salt Company in the bed of the Salton Sea were already entirely submerged, the water covering about 100 000 acres, with a maximum depth of about 6 ft. Except for the increase of depth and the consequent increase in the length of time this property would be shut down, no additional damage was really being done at this point. Indeed, 14 years earlier, this property was covered to a depth of 6 ft. by the great flood of February, 1891, and the summer flood following, and in all probability a similar and greater inundation would have resulted from the excessive floods during the spring and summer of 1905 had the C. D. Co. never constructed any works along the river. The rising waters of the Salton Sea were threatening the tracks of the Southern Pacific Railroad along the east side of the sink, and the officials of the Los Angeles Division were clamoring for aggressive action. The higher officials of the company, however, were not yet very much perturbed. On the other hand, the Alamo channel was being enlarged and deepened, to the very great benefit of the C. D. Co., and the irrigation system of Imperial Valley, because the insufficient carrying capacity of this channel and the heavy silt deposits therein constituted a serious menace to the entire project.

To close the lower intake entirely meant obtaining all the water required for the irrigation of Imperial Valley through the 4 miles of badly silted Main Canal lying between it and the upper intakes, and this was out of the question. Even with large sums of money, which might be obtainable from the Southern Pacific interests, machinery could not have been bought, assembled, and put into operation in time to have permitted the delivery of more than enough water to supply the inhabitants and live stock of the valley with drinking water if the river flow should be reduced to 6 000 or 7 000 sec-ft. Imperial Valley at that time consisted of at least 125 000 acres under cultivation, five

towns with an aggregate population of 2 500 people, and a rural population of approximately three times that number. There were, perhaps, 100 000 head of hogs, 50 000 head of cattle, and other live stock in proportion.

Many plans were suggested, from this time, August 1st, 1905, until the break was finally closed in 1907. Many of these, of course, were thoroughly absurd, and came from cranks and people who had not the faintest conception of the conditions. Indeed, almost the only people who appeared to be able to see that the problem was not merely one of shutting off the lower intake were the engineers of the C. D. Co. and a few of the well-informed men in Imperial Valley. Representatives of the New Liverpool Salt Company, the Southern Pacific Company, various departments of the United States and Mexican Governments, and the general public, all joined in demanding aggressive action to stop the menace of a new Salton Sea.

Such suggestions were addressed to Mr. Harriman and to nearly every other official of the Southern Pacific interests, and to Mr. Randolph and other authorities of the C. D. Co. Ultimately, most of these found their way to the writer; they constitute a most interesting collection. It is not profitable to mention more than four of these suggestions, which may be designated the Laguna Weir Plan, the Concrete Head-gate Plan, the Rockwood Head-gate Plan, and the Barrier Dam Plan. Edwin Duryea, Jr., M. Am. Soc. C. E., also offered to close the break according to a plan, which, however, he declined to outline.

The Laguna Weir Plan.—The Laguna Weir Plan consisted in abandoning operations for the time being at the scene of the break; concentrating all efforts on the completion, at the earliest possible date, of the Laguna Weir, which was being built by the U. S. Reclamation Service; building a canal thence passing Pilot Knob and intersecting the break from $\frac{1}{2}$ to $\frac{3}{4}$ mile west of the Colorado River, this canal to have a capacity equal to the low-water flow of the river; then diverting all the river water through this canal; finally, to build a dam across the intake between the canal junction and the river bank in still water. The Laguna Weir was actually completed in the early spring of 1909, just before the annual record flood of that year. It is not clear just how its completion could have been essentially hurried. Had this plan been followed, the Colorado would have emptied into the Salton

Sea for 3 years longer than it actually did, and during this time 55 000 000 acre-ft. of water went by Yuma, only a very small portion of which would have gone down the old channel to the Gulf. This would have raised the water in the Salton Sea to the 180-ft. contour, with the effect of drowning out a large area of cultivated land in the Coachella Valley and forcing the abandonment of 60 miles of main line track by the Southern Pacific Railroad.

These effects, however, would have been of relatively minor importance. The irrigation system of Imperial Valley would have been strained far beyond the breaking point in several places, while the cutting back in New River would unquestionably have reached the Alamo channel and lowered the water therein far beyond the point where any could have been gotten into the Imperial Valley by gravity. This, of course, would have meant the depopulation of that region, an appalling result, without parallel in history.

The Laguna Weir Plan is thus seen to have been impracticable, and no one actually connected with the work gave it serious consideration. Nevertheless, Director Walcott, of the U. S. Geological Survey, who was a close personal friend of Mr. Harriman, strongly advised it, and Mr. Harriman almost insisted upon it for quite a time.

Concrete Head-gate Plan.—The Concrete Head-gate Plan was put forward by the late James D. Schuyler, M. Am. Soc. C. E., who acted as Consulting Engineer of the C. D. Co. from July, 1905, to June, 1906. It consisted essentially of building a reinforced concrete and steel head-gate on the Pilot Knob site, where solid rock foundation could be secured, such gate to be able to carry the low-water flow of the river; and then, from this head-gate down to its junction with the crevasse, to enlarge the canal to a similar capacity. This, it was considered, would permit the diversion of all the water through the head-gate and canal, leaving the river below, and consequently the break itself, dry. The underlying idea was somewhat similar to that of the Laguna Weir Plan, except that it contemplated only 4 miles of canal enlargement and a diversion structure which could be completed in 3 or 4 months, instead of 3 years.

This plan involved the construction of permanent head-gates on rock foundation at Pilot Knob, so long contemplated; and the construction and equipment of a dredge with which the requisite 4 miles of canal could be dug economically and quickly. The idea was adopted

in a tentative way in September, 1905, approximately 90 days after the Southern Pacific Company undertook the management of the C. D. Co., and Mr. Schuyler was instructed to proceed with the preparation of plans for the head-gate, while Mr. F. S. Edinger, under whose direction the Edinger Dam was built, arranged for the dredge. At the suggestion of the Golden State and Miners Iron Works, of San Francisco, the clam-shell type, with 150-ft. boom and 5-cu. yd. bucket, was selected. Work was begun on the concrete head-gate on December 15th, 1905, and contracts for the clam-shell dredge were arranged a few weeks later.

One of the chief recommendations of this plan was that the constructions, in large measure, would be permanent. It was assumed that, while perhaps the maximum quantity of water which would have to be diverted for the irrigation of Imperial Valley would never exceed 5 000 sec-ft., a gate twice as large would not have any particular disadvantages in its maintenance or operation. It was urged, further, that this arrangement of diverting structure and large canal would be available in case of future breaks, should any ever occur.

The difficulty about the plan was that, regardless of the size of the gate, enlarging the 4 miles of canal to carry 10 000 sec-ft. within sufficient time to afford reasonable relief was a very serious problem, while the capacity of this canal would be reduced so quickly by silt deposition that its use in case of future breaks would be out of the question. Furthermore, to insure the diversion of all the water in the river, required a canal cut considerably below the water-table in the ground through which it would have to pass, and large patches of quicksand occur so frequently in this region that it would be folly to hope to miss all of them. Such patches would cause the inflow of material from the sides and the bottom to a serious extent.

Mr. Rockwood's Plan.—Mr. Rockwood urged the necessity of a rapid re-diversion, not so much because of the effect on the Southern Pacific tracks along the Salton Basin as because he understood the critical condition at a number of points in the irrigation system of Imperial Valley, and that the severe strain could not be withstood successfully for very many months. His suggestion, made in August, 1905, was to put in, immediately beside the break, a wooden **A**-frame, flash-board head-gate, capable of passing the low-water flow of the river; with dredges to dig channels from the break to the gate both above

and below; divert the water through this by-pass and gate with a piling-brush-sandbag barrier dam; complete the dam as an earth fill across the break, and build levees both up and down stream as far as might be necessary; then close the gate to such an extent as would admit only enough water to supply the irrigation needs. This plan was approved, and work was started on September 20th. It was abandoned completely 3 weeks later; was again approved on December 15th, 1905; and was carried out until the gate construction failed, in October, 1906. It was daring only in its size and the foundation of so important a structure on alluvial soil, and it would have resulted in permanent diversion works on Mexican soil—where, by all means, they should have been, originally, and as contemplated in the Mexican concession, granted in 1904.

The Barrier Dam Plan.—The Barrier Dam Plan consisted in throwing a barrier dam of some sort across the crevasse and raising the water surface above it sufficiently high to throw all the discharge of the river down the old channel to the Gulf. The usual type of dam suggested was of piling and brush mattresses of fascines weighted down by sandbags. This method seemed to its proponents to afford opportunity for decreasing the quantity of water diverted in the minimum time, and neglected that side of the problem which required the furnishing of water for the Imperial Valley. The best plan for a structure of this type was that put forward by Mr. Edinger, and worked on under his direction from early in October until its destruction by the great flood of November 29th, 1905.

Fourth Attempt to Close the Crevasse.

Mr. Rockwood presented his plan to Mr. Randolph and Mr. Schuyler, and they, as well as several engineers of the Southern Pacific Company, approved of its trial. Plans were hurriedly worked out for a wooden A-frame, flash-board gate, 120 ft. long, with a concrete floor, and founded on piles. Rush orders for materials were placed, and the first shipments left Los Angeles on August 7th. It was fully expected to have the structure completed by November 15th.

The original intention was to construct the gate in a by-pass to be excavated by the dredge *Alpha* on the south side of the intake, but examination showed an unfavorable foundation, as the ground slid into the cut so rapidly that the dredge was almost caught and held by it. The plans, therefore, were changed, and it was decided to construct

a by-pass on the other side of the break; force all the water through this by-pass; and then build the structure where the intake had been, thus saving both time and money in the excavation. The break at this point was about 300 ft. wide—just about the length of excavation required for rapid and successful construction. The dredge was put to work on this by-pass, and no difficulty whatever was found in making the 700-ft. cut required. The plan worked very well, and a large part of the water began to go that way at once. Work was begun on the up-stream side of the coffer-dam, the idea being that, when all the water was diverted through the by-pass, another earthen dike would be thrown in, about 250 ft. below the first, and thus make the coffer-dam for the gate construction. In this way, the second dam would be built in still water and in very short order with the dredge.

At this time—about September 15th—it became evident to Mr. Rockwood that he could not attend to the business affairs of the company properly and remain in personal charge of the work along the river. It seemed easier to find some one capable of completing the gate in accordance with the plans outlined than to find any one qualified to handle the corporation's affairs. Mr. F. S. Edinger was selected, as he, until June, 1905, had been for many years Superintendent of Bridges of the Southern Pacific System, and had had very large experience in coffer-dam work. About 3 months previously he had left the Southern Pacific Company and entered the contracting firm of Shattuck and Desmond, of Los Angeles and San Francisco. About September 20th, Mr. Edinger and Mr. Rockwood went over the ground and the plans together, and Mr. Edinger commenced the work.

The records show that, about October 1st, the river usually rises 2 or 3 ft., principally due to rains on the water-shed of the Gila River and Bill Williams Fork. This year was no exception, and the slight rise about October 1st shook Mr. Edinger's confidence in the plan. He quickly outlined a barrier dam plan to Mr. Randolph, who approved of it, and work was shifted to it at once. This plan consisted of constructing a piling and brush dam across the west channel between the head of Disaster Island and the Mexican shore, a distance of about 600 ft., and it was expected that the river would all be turned down the east channel before a gate could even be put in. All material was at once removed from the gate site, and work was rushed on the construction of what is locally known as the Edinger Dam.

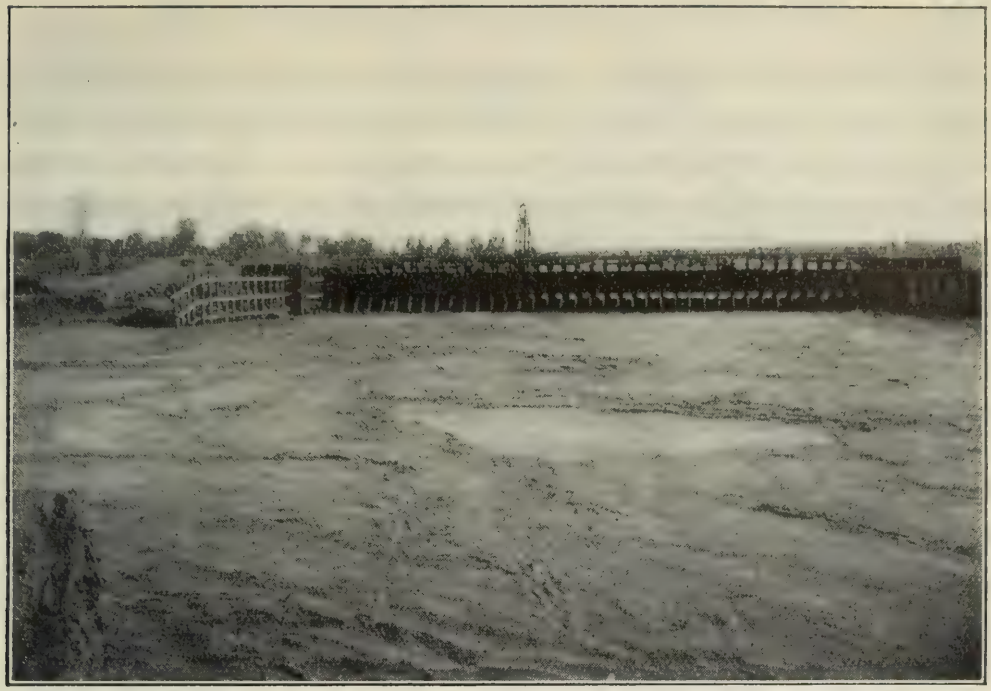


FIG. 19.—ROCKWOOD HEAD-GATE, OCTOBER 6TH, 1906, PASSING 12 000 SECOND-FEET.

FIG. 20.—EDINGER DAM, NOVEMBER 8TH, 1905. BRUSH AND WIRE MAT IN FOREGROUND, READY TO BE PLACED.

This plan of handling the situation, in addition to shutting off all water flowing into the Imperial Valley through the lower intake, was seriously defective in that even a short flood sufficiently great to send any water overbank in the immediate vicinity of the dam—and that would require much less water than usual on account of the silted-up condition of the whole river bed below the break—would in a few hours result in cutting a channel around the end of the structure and entirely shunting it. Indeed, such a re-diversion was exactly what took place a little more than a year later, when the waters broke under the levee, ½ mile south of the Hind Dam, in December, 1906. Had the Edinger Dam been entirely successful and completed on November 15th, such re-diversion would have been caused by the terrific flood of November 28th, and so on; the hydrograph, Plate XLVII, shows a number of floods sufficiently great to have done this. Indeed, at this time, no one seems to have realized that a large, deep, and efficient channel had been created from the Lower Heading westward for many miles, and that future safety demanded, not only closing the intake, but an elaborate system of levees reaching miles both up and down stream.

The plan of the Edinger Dam consisted in driving rows of piling and filling the interstices with brush mattresses and fascines. The idea behind it was essentially similar to that of the work abandoned about August 1st. To have been successful, the construction would have had to withstand a head of from 8 to 10 ft. However, on November 29th, when a head of 35 in. had been obtained, a terrific flood came down from the Gila, reaching a gauge height of 31.3 ft. at Yuma and a discharge of 115 000 sec-ft. Large quantities of drift were carried by the flood-waters. This drift collected against the Edinger Dam in great quantities, and a large volume of water went down the east side of the island and the old channel. Before the flood had reached its peak, the dam started to give way, and in an incredibly short time was practically destroyed. When the river had again fallen, the old channel was silted up higher than before, the new channel was scoured still deeper, and when the flow of the river had decreased to 17 500 sec-ft. all the water was again going toward the Salton Sea.

The flood not only wrecked the dam, but carried away practically all the material on the ground, and, after it receded, side-cutting along the west side of Disaster Island began to take it away rapidly. It was soon obvious that it would be folly to resume work at that location,

and it was decided that the piling-brush-sandbag barrier dam method was not to be given further consideration.

So much water went through the break to the valley, at the failure of this dam, that the *Alpha* was sent to the Quail River and put to work cutting a channel southward in the hope of diverting a large part of the flow into the Paredones and thence *via* Volcano Lake into the Gulf. It was an endeavor to divert a large part of the water from an old overflow channel on the north side of the delta cone into an overflow channel on the south side thereof. It had little result, however, and the Quail River cut soon closed itself.

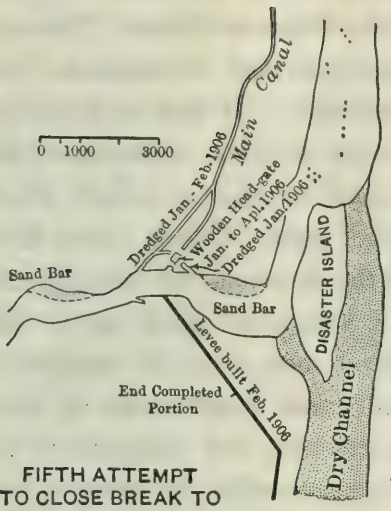
On October 15th there were 20 white men and 25 Indians at work on the Edinger Dam; on November 1st, 42 white men and 50 Indians; on November 10th, 106 white men and 65 Indians, and on November 29th, 250 white men and 80 Indians. Two steamboats with barges attached, and the relatively large barge, *Silas J. Lewis,* with their crews, were also at work.

On the books of the company, the cost of the Edinger Dam is not thoroughly segregated from the expense incurred in the head-gate work up to the time of its abandonment for the barrier dam plan. The expenditures on it, however, were about $60 000, and the grand total to December 1st, 1905, was about $100 000.

Fifth Attempt to Close the Crevasse.

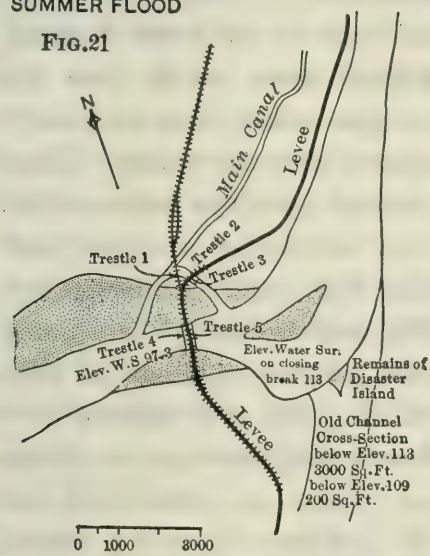
It was decided to work simultaneously on the Concrete Head-gate plan and the Rockwood plan, which was done.

Concrete Head-gate.—The location of this interesting structure is shown on Fig. 22, where the granite point of Pilot Knob is near the right bank of the river. The general design was outlined by Mr. Schuyler, and the principles used, dimensions, elevations of flow, etc., were submitted to Messrs. Rockwood and Randolph, and approved by them. George S. Binckley, M. Am. Soc. C. E., then worked out the details and prepared the working drawings. Contracts for the structural steel and ironwork were let to the Llewellyn Iron Works, of Los Angeles, and for the construction work to Mr. Carl Leonardt, also of Los Angeles, on November 25th, 1905. Time was made the essence of the contract, and Mr. Leonardt hurried the necessary equipment to the ground and began actual work 2 weeks later. Although it was expected to complete the gate ready for use within 90 days, the entire job was not finished until June 28th, 1906.



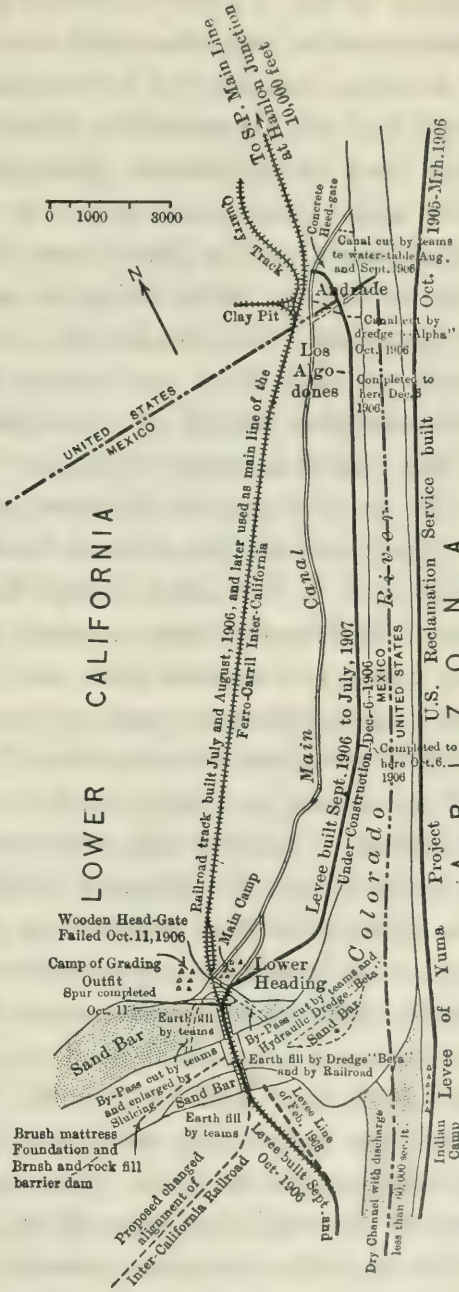
FIFTH ATTEMPT TO CLOSE BREAK TO APRIL 15, 1906-BEGINING OF SUMMER FLOOD

FIG.21



SIXTH AND SUCCESSFUL ATTEMPT TO CLOSE THE BREAK OCT. 12 TO NOV. 4, 1906

FIG.23


0 1000 3000
To S.P. Main Line at Hanlon Junction 10000 feet
Quarry Track
Concrete Head-gate
Canal cut by teams to water-table Aug. and Sept. 1906
Andrade
Clay Pit
Canal cut by dredge "Alpha" Oct. 1906
Los Algodones
Completed here Dec. 6 1906
UNITED STATES
MEXICO
LOWER CALIFORNIA
Railroad track built July and August, 1906, and later used as main line of the Ferro-Carril Inter-California
Main Canal
Levee built Sept. 1906 to July, 1907
Under Construction Dec. 6, 1906
Completed to here Oct. 6, 1906
Colorado River
U.S. Reclamation Service built Oct. 1905-Mch. 1906
Project
ARIZONA
Wooden Head-Gate Failed Oct. 11, 1906
Main Camp
Camp of Grading Outfit
Spur completed Oct. 11
Lower Heading
Earth fill by teams
By-Pass cut by Hydraulic Dredge "Beta" and enlarged by Sluicing
Sand Bar
Earth fill by Dredge "Beta" and by Railroad
Brush mattress Foundation and Brush and rock fill barrier dam
Levee Line of Feb. 1906
Levee built Sept. Oct. 1906
Proposed changed alignment of Inter-California Railroad
Indian Camp
Dry Channel with discharge less than 50,000 sec.-ft.

FIFTH ATTEMPT TO CLOSE THE BREAK-JULY 1, TO OCT. 11, 1906 AFTER SUMMER FLOOD

FIG.22

Type and Size.—The intake gate is doubtless the largest and most expensive irrigation canal head-gate in America. The design is a modification of the Taintor or radial-gate type, which has been used for many years for irrigation constructions in the Western States. This style of structure was adopted in order to obtain openings of maximum area easily and quickly opened or closed by one man. It had probably not been used in California, although a large wooden radial-gate had been built some years before at the head of the so-called Peoria Canal from the Gila River, near Gila Bend, Ariz. It was about 25 ft. high and 30 ft. wide, which is nearing the extreme for construction of that class. This wooden gate, however, was never used, as the dam across the Gila River was destroyed by flood soon after its completion. The maximum height of radial-gates and canal head-works in Idaho at the time was about 11 ft., and the water was not expected to rise to the top of the gates, the river level being controlled by other means.

Here, however, the extreme flood level is 19 ft. higher than the low-water level, so that gates at high flood time are subjected to great pressure. Sufficiently large vertical lifting gates would have required very heavy and massive piers, and the gate would have been very large and disproportionately high as compared with the width. These considerations caused the adoption of culvert openings between the piers for supporting a cellular structure of reinforced concrete, and thus admitting of loading the construction with gravel filling in the cells in order to get the required stability and weight at minimum cost. The gates were thus required to close culvert openings of minimum size, being in fact no larger than with the head at a uniform low-water height, although, of course, much heavier and stronger on account of the increased pressure at flood stages. There are eleven such culverts, each 10 ft. high and 12 ft. wide. In addition, there is a "navigation pass," the purpose of which was to permit passing a small gasoline launch through the gate. This navigation pass is practically useless because the mill race through it, when the difference in water level above and below the gate exceeds 1 ft., precludes the idea of dragging a boat through it; indeed, no attempt has ever been made to use it. The floor of the gate is 98 ft. above sea level, according to the C. D. Co. datum, and 100.9 ft. according to the U. S. Reclamation Service datum. At the time, and until after the summer flood of 1909, the average low-water surface in the river was about 108 ft. The elevation

of the flow line at the gate, therefore, was fixed so that the culverts would run full at low-water stage. The present low-water surface is about 105 ft.

The area of the eleven culverts is 1 320 sq. ft., and, with the water 1 ft. higher on the up-stream than on the down-stream side of the gate, their combined discharge would be 8 500 sec-ft. In addition, a large quantity of water would go through the navigation pass, which is 10 ft. 3 in. wide. When the water is 10 ft. above the top of the culverts, it is necessary to close the gates within 3.8 ft. of the bottom to hold the discharge through them down to 10 000 sec-ft., when the carrying capacity of the canal below is great enough to allow the water to get away.

The gate was designed to pass the entire low-water flow of the river—which it was assumed would certainly not exceed 10 000 sec-ft.—without any diverting dam in the river opposite it.

Cost of Structure.—Table 12 gives the cost of this structure, with the contract prices for excavation, concrete work, etc.

The cost of the gate, however, was considerably more, because Contractor Leonardt presented a claim, insisting that the prices for earth and rock excavation named in his contract were agreed to by him on certain assurances made by Mr. Rockwood as to the character of the excavation, which had proved more difficult than was expected. This claim was made as soon as Mr. Leonardt's representatives reached the ground, and Mr. Randolph permitted a change to a force account basis because of his desire to hurry the construction in every possible way. The earth excavation in this way cost 64 cents per cu. yd. and the rock $2.06, thus increasing the figures by $10 813, making a grand total of $55 221.08.

Careful accounts were kept, and it was ascertained that the contractor made a profit of $2 700 on the concrete, and $741.50 on erecting the gates. What the earth and rock excavation should have cost is a matter of some, though slight, interest to the Profession, as these would necessarily vary according to local conditions. As a matter of fact, with a good pumping plant, a mining nozzle or giant, a hydraulic elevator, and some pipe, the earth excavation could probably have been handled for 20 cents per cu. yd., and possibly less. Much of the rock was fairly soft, and could have been worked easily and cheaply, so that, had the contractor put in power drills and one or two long-

boom derricks to handle the rock out of the cut, it is probable that the cost of such excavation would not have exceeded the contract price. The quantity of water entering the coffer-dam, or rather excavation pit, was surprisingly small.

TABLE 12.

Gate Structure.	
Earth excavation, 12 637.1 cu. yd. at $0.25	$3 159.28
Rock excavation, 5 700.81 cu. yd. at $1.00	5 700.81
Cement, furnished by company, 1 335 bbl. (Olsen, Gillingham, and Independence brands)	4 432.25
Concrete, labor, forms, sand, gravel, and rock, 1 204.83 cu. yd. at $9.00	10 843.47
Reinforcing steel bars, 25 722 lb. at 4 cents	1 965.16
Expanded metal for gate facings, 791 lb. at 4 cents	31.64
Allowance for 3 days' delay to contractor	102.50
Extras	807.07
	$27 042.18
Charges against contractor	271.70
Total cost of gate structure	$26 770.48
Iron and Steel Work for Gate.	
Llewellyn Iron Works' original contract for twelve radial gates and one slide-gate (in navigation pass) f. o. b. Los Angeles	$12 000.00
Freight to Yuma, on 212 184 lb. metal in aforesaid gates at $1.25 per ton	132.60
Regulating levers, shaft, and gear (subsequent contract)	580.00
Erection of gates (Leonardt's contract)	1 500.00
Total cost of iron and steel work	$14 812.60
Plans, engineering, and superintendence, 6.7%	2 825.00
Total cost of head-works	$44 408.08

Fig. 26 gives a plan and elevation of this gate, and Figs. 24 and 25 are views of the structure.

Purpose.—At the time this gate was designed, the money available for construction, through the Southern Pacific's connection and the loan of $200 000, justified the immediate construction of permanent head-works, indeed, building this and the Alamo Waste-gate were the chief items for which the loan was made. Furthermore, the entire diversion of the river at the lower intake had shown the folly of trying to get along without them. The gate, while intended as a permanent diverting structure, was, nevertheless, primarily designed for use in endeavoring to re-divert the river; otherwise, of course, it

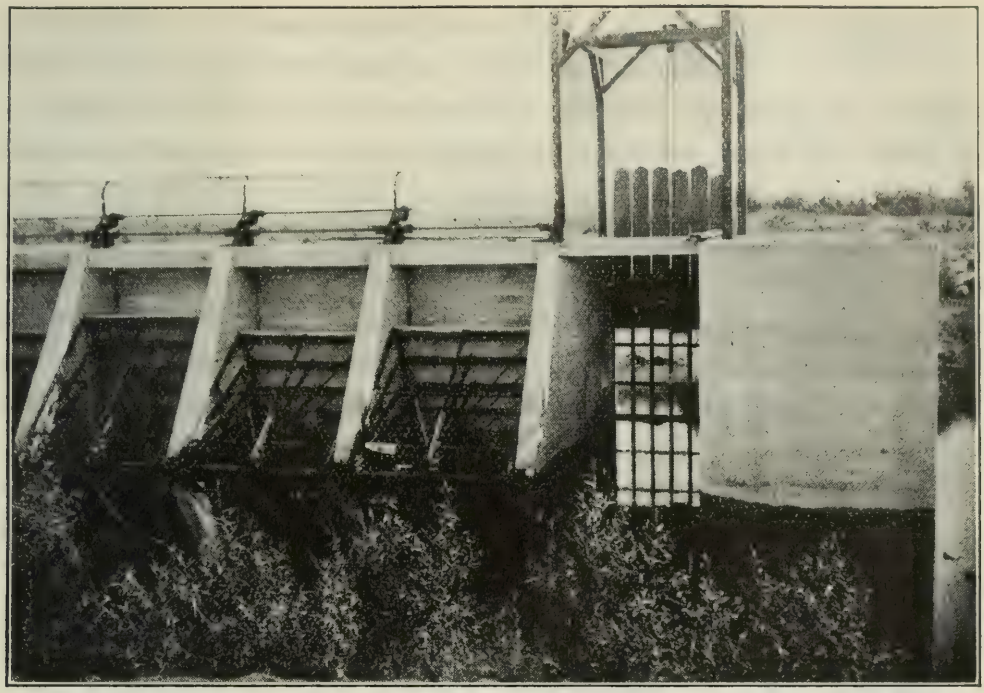


FIG. 24.—CONCRETE HEAD-GATE, JULY 10TH, 1906, SHOWING DETAILS OF GATES, NAVIGATION PASS, AND GATE AND ABUTMENT AT RIVER END.

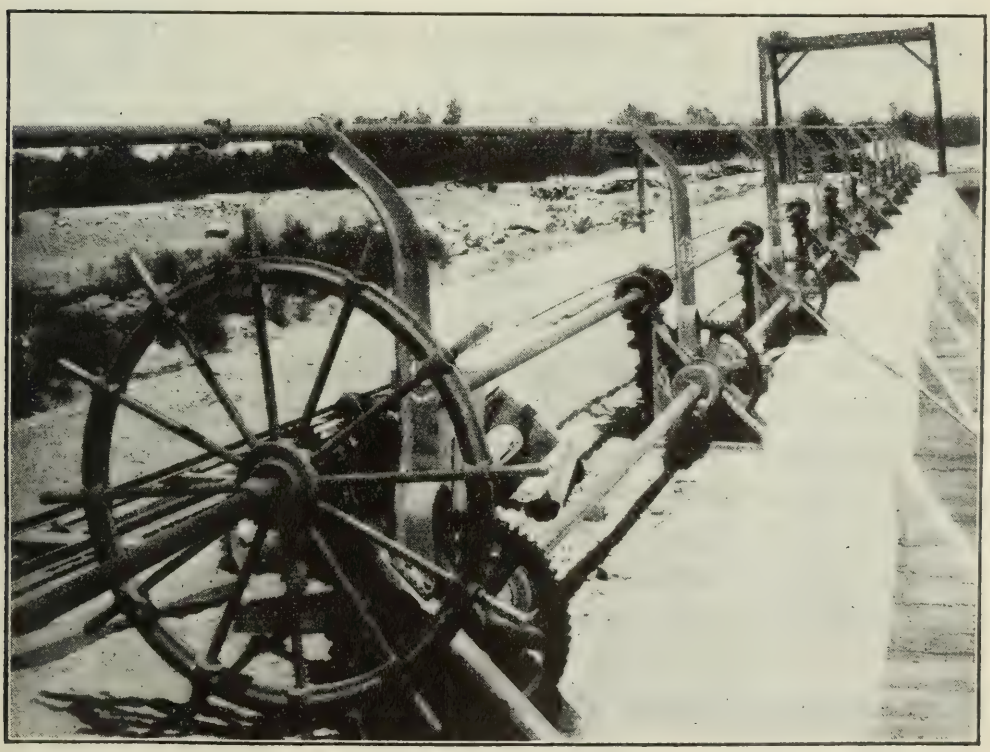
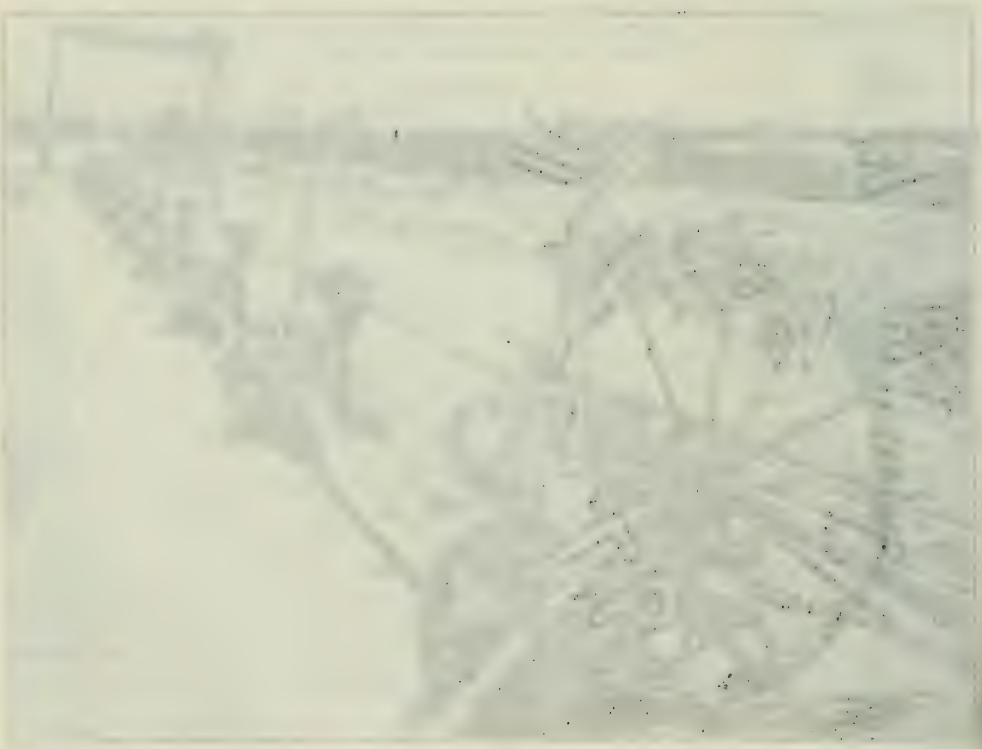


FIG. 25.—GATE RAISING MECHANISM OF CONCRETE HEAD-GATE.



would have been made much less than half as large. Actually, it played no part at any time in diverting the stream.

Operation.—The gate was actually put into operation on November 1st, 1906, when the water going through the break had been reduced to a quantity too small for the requirements of the valley. About 6 weeks later, the flood which caused the second break occurred, and resulted in an accumulation of drift on the up-stream side of the gate which choked up the underground culverts and practically put it out of commission. From that date to this the troubles caused by drift in the river, particularly at high-water periods, have been serious

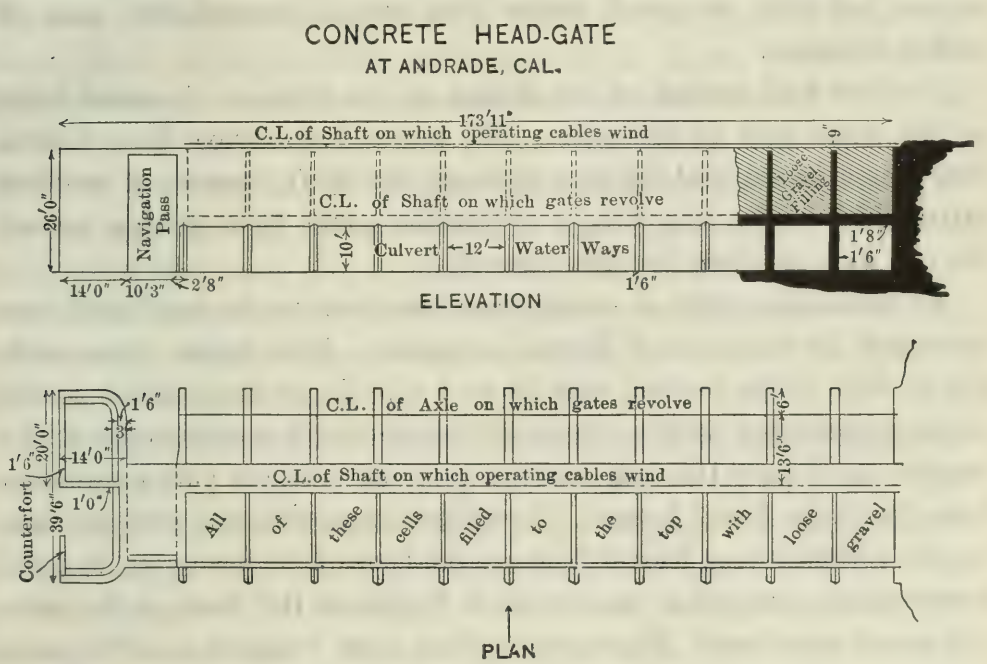


FIG. 26.

and often acute. Gates of this type, for head-works on a river carrying any drift to speak of, let alone as much as the Colorado often has, should be avoided. After considerable experience it is obvious that if permanent diversion of the water for the irrigation of the valley is not made on Mexican territory, then, whenever enough money is available, it will be best to abandon the structure entirely and make diversion through gates similar to those in the sluice-ways of the Laguna Weir.

Aside from the type of gate for such a locality and stream, three unfortunate features in design became manifest. Chief of these was the fault that the drums on which the wire cables for raising the

gates are wound are much too small. The gates themselves were designed for minimum weight with the necessary strength, and are not stiff enough, so that they tend to wedge unless exceedingly great care is taken. The net result is many broken cables. At one time only two of the eleven gates were in operation, some being clear down, some clear up, and some impossible to close entirely on account of driftwood under them. Fortunately, the *Delta* was near by and was used to raise the gates, so that new and strong plow-steel cables could be installed, replacing the original ones of Tobin-bronze, ¾ in. in diameter, 19 wires to the strand. These plain steel cables corrode badly, of course, but still are much better than any galvanized iron ones of usable diameter.

Another bad feature of the design is the form of abutment built on the outer end of the gate. The writer has always been fearful that water would find its way through the 10-ft. tongue of puddled earth which is the only barrier preventing water from getting around the end and shunting the gate entirely.

In September, 1906, a canal, from the river to the head-gate, was excavated by teams and Fresno scrapers. This intake was made 100 ft. wide at the bottom, with 2½ to 1 side slopes down as low as the underground water-table would permit. At about the same time the *Alpha* reached the Upper Heading and cut into the concrete gate excavation from the Main Canal below. The upper connection was wide enough, but the bottom was at least 6 ft. above the floor of the head-gate, and the down-stream connection was about 3 ft. above the floor of the gate and much narrower. These connections were widened and deepened to their present capacity by erosion, dredging, and blasting, as explained later.

The Dredge, Delta.—The other element in the concrete head-gate plan of re-diversion was a canal from the head-gate to the break, a distance of approximately 4 miles. It was to be of sufficient size to carry the probable minimum flow of the river, 10 000 sec-ft. As it was obvious that this stretch of canal would have to be lower than the bed of the river all along the line, in order to permit of taking the entire low-water flow without a diversion dam in the river opposite the head-gate, a very large part of the cross-section to be excavated would be below the permanent water-table of the region. Therefore, some kind of excavating machinery which could handle large quantities of ma-

terial under water had to be provided. It was taken for granted that the cheapest and quickest method of providing this waterway was to enlarge the existing Main Canal, although the writer thinks this was erroneous. The dipper dredge, *Alpha,* by almost continuous operation in this part of the course, had built up levees on both banks so high as practically to limit its future operation without flattening down these levees with teams and scrapers. Largely on the advice of Mr. Edinger, it was decided to construct a clam-shell dredge of the type used almost exclusively for levee building along the Sacramento River. Accordingly, a contract for machinery, and for plans, bills of materials, etc., of the hull, was entered into with the Golden State and Miners Iron Works, of San Francisco, which makes a specialty of clam-shell dredge machinery, construction, and even operation, on the Pacific Coast. This contract was closed on January 10th, 1906. The 320 000 ft. of Oregon pine lumber and other materials for the hull were bought through the purchasing department of the Southern Pacific Company, and the unusually large timbers required were obtained in Oregon and sent directly to Yuma. In the purchase of both hull material and machinery, time was considered as of the essence of the contracts.

A dredge with a 150-ft. boom, carrying a 5-cu. yd. bucket was decided on, and a hull 120 ft. long, 54 ft. wide, and 11 ft. deep. This width was 2 ft. greater than had ever been built on the Coast, although the tendency is to increase the dimensions, and one is now building in San Francisco, 70 by 140-ft. hull, 205-ft. boom, and 6-cu. yd. bucket. The machinery is a 150-h.p., internally-fired, circular, fire-tube boiler, and a 20 by 24-in. engine on each side. It was decided to build the hull and erect the machinery at Yuma, and float the completed dredge down the river to the intake.

Lumber for the hull began arriving in Yuma late in January, and early in March the company was notified that all the machinery was ready at San Francisco for shipment. Mr. Edinger's connection with the company had ceased soon after the destruction of the Edinger Dam, and Mr. Rockwood had very little confidence in the feasibility of the concrete head-gate plan, or in the desirability or need for the clam-shell dredge, and felt that the great cost thereof would deplete seriously the $200 000 loaned by the Southern Pacific Company. Therefore, practically nothing was done in the matter, and so it came about

that the great conflagration in San Francisco, following the earthquake of April 18th, 1906, destroyed the plant of the Golden State and Miners Iron Works, in which all the machinery for this dredge was stored ready for shipment. Fortunately, the damage sustained by the apparatus was not extensive, and by May 15th, 1906, all the machinery had reached Yuma.

Mr. J. W. Brown, a member of the Golden State and Miners Iron Works Corporation, agreed to take charge of building the hull, and reached Yuma about May 1st, bringing with him a complete crew of mechanics and ship builders. Work was hurried, and with such success that the hull was launched about August 15th, the machinery was in place by the end of October, and the dredge weighed anchor and started down the river. At this time the river was getting low and some difficulty was encountered, but on November 26th, 1906, the clam-shell arrived at the mouth of the American intake. The total cost of the dredge, ready to start down the river, was almost $80 000, the cost of the machinery being $34 000, f. o. b. San Francisco. The weight of the craft is about 850 tons.

This dredge, Plate XLVIII, has been an invaluable piece of machinery to the C. D. Co. Had it been ready for use in August, 1905, the cost of doing the earthwork in the Hind Dam would have been wonderfully reduced. As it was, the dredge, after doing a little work in enlarging the intake above the concrete head-gate, was floated down and cut its way into the Main Canal following the upper Mexican intake. It was engaged on this work when the second break occurred, and continued thereon as though this latter event had not happened. Like the concrete head-gate, it played no part whatsoever in the rediversion of the river.

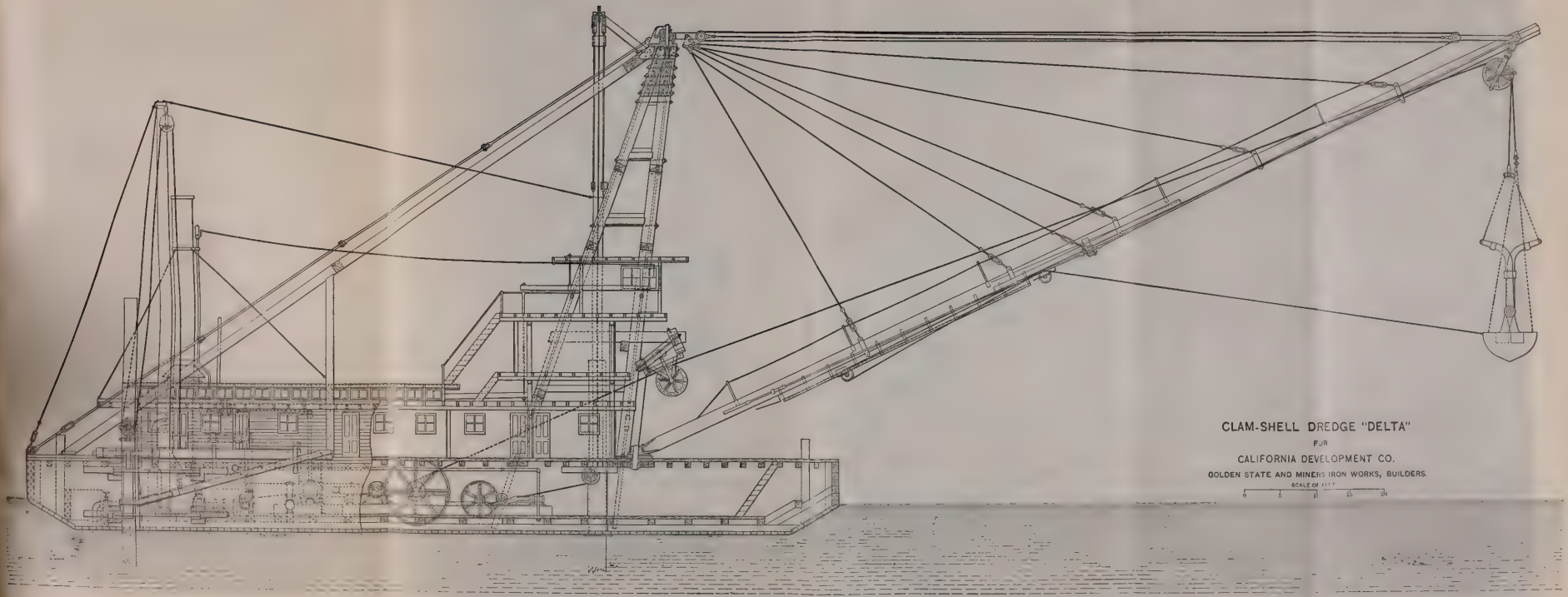
For the information of those who are not familiar with the results and cost of clam-shell dredge operation, the following data are given:

Operatives.

1 Captain	at	$125 to $150	per	month,	and	board.
3 Levermen	"	85	"	"	"	"
2 Firemen	"	60	"	"	"	"
2 Deckhands	"	50	"	"	"	"
1 Cook	"	50	"	"	"	"
1 Blacksmith	"	90	"	"	"	"
1 Roustabout	"	40	"	"	"	"

PLATE XLVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1279.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.

1512-A.



CLAM-SHELL DREDGE "DELTA"
FOR
CALIFORNIA DEVELOPMENT CO.
GOLDEN STATE AND MINERS IRON WORKS, BUILDERS.
SCALE OF FEET
0 5 10 15 20

Three shifts give a total of 22 hours actual work daily. The average time in operation, when proper repair work is done, is 28 days per month. When in good ground, and with side swings averaging 70° on each side, the time per bucketful is 40 sec. The quantity handled (varying according to the material) is from 3 to 8 cu. yd. as ordinary extremes. On the Sacramento River, under good conditions, 150 000 cu. yd. per month are handled.

Monthly expense.

Maintenance and operation	$2 500
Interest on investment at 6%	400
Taxes and insurance	200
Deterioration	700
Sometimes as low as	$3 800

Ordinarily, the monthly expense in Mexico is $5 000. The average cost is 2¾ cents per cu. yd. The average cost of the work done by the *Delta* in Mexico is from 4 to 6 cents per cu. yd.

The Rockwood Head-Gate.

As already explained, it was decided to follow both Mr. Schuyler's and Mr. Rockwood's plans for diverting the river, and so, for the second time, on December 15th, 1905, Mr. Rockwood was authorized to proceed with the construction of a wooden head-gate beside the lower intake. The heavy flood of November 29th and its receding waters had widened the intake from 300 to approximately 600 ft., and, after considering the conditions, it was decided to build the gate directly in the old canal about 200 ft. north of the intake channel, in order to reduce the time and the quantity of excavation required, and to divert the relatively small quantity of water in the old canal around the gate with a by-pass to be dug by the *Alpha*. The gate, started and abandoned three months before, was originally planned for a width of 80 ft.; this was increased to 120 ft. in order to carry a maximum of 9 000 sec-ft. As the gate could not be completed until the spring of 1906, the length was extended to 200 ft. The over-all dimensions, including the wooden aprons, became 240 by 100 ft. The entire space, of course, had to be inclosed in a coffer-dam, and the excavation had to be made inside of it. The plans are shown on Plate XLIX.

As far as the writer has ever heard, this is the largest and most daring design ever made for a wooden **A**-frame, flash-board head-gate. Pile-driving was begun on January 7th, and the gate was completed on April 18th, 1906; the work was rushed day and night for the greater part of the time, and no real difficulties whatever were encountered. As in the case of the concrete head-gate, 4 miles above, the quantity of water seeping into the excavation was surprisingly small. The various items of the cost of this structure were not segregated, so that the details cannot be given, but the grand total expense of the gate proper, exclusive of the by-pass, was approximately $122 500.

The discharge of the river by April 10th, was 32 200 sec-ft., and showed that the annual flood had begun, therefore all idea of attempting to divert the water through the gate by damming the crevasse itself before the summer flood should have been passed, was abandoned.

Change in Engineering Staff.—On May 15th, 1905, the writer was transferred from the Southern Pacific Company in San Francisco to the Associated Harriman Lines in Arizona, with the title of Assistant to the President of those properties. About 5 weeks later Mr. Randolph's duties were increased, as he was put in charge of the C. D. Co. and the Mexican Co., and shortly thereafter the officials of the Southern District of the Southern Pacific Company urged on him the very serious fact that the track beside the Salton Sea would soon be under water, and insisted that aggressive action be taken to close the break on the river. About the middle of July Mr. Randolph sent the writer to the river to confer with Mr. Rockwood, and a day was spent together examining the situation. About August 1st a second trip was made, and after the disastrous flood of November 28th, 1905, a third visit. Toward the end of January Mr. Randolph again sent the writer to the Lower Heading to assist Mr. Rockwood in hurrying the construction of the wooden head-gate. As this work neared completion, Mr. Rockwood suggested that he had found it impossible to handle things in accordance with his own ideas; he believed that the best interests of all concerned pointed to his resignation, and urged that the writer take up the work. After considerable discussion it was agreed that, if Mr. Randolph also desired the arrangement, there would be no objection offered. Shortly afterward these gentlemen met in Los Angeles and agreed to the change, and on April 19th, Mr. Rockwood

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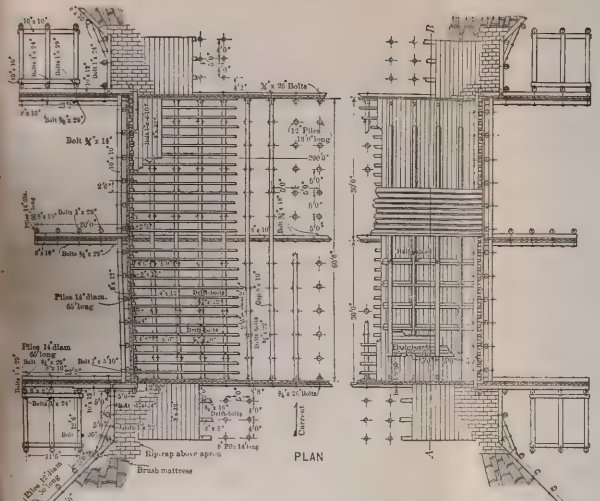
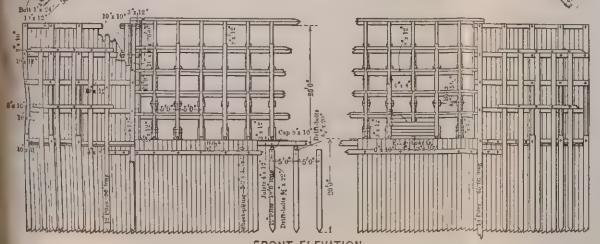
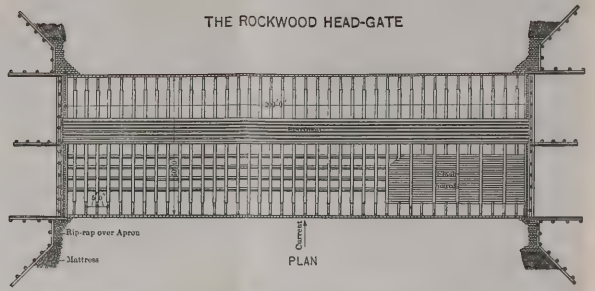
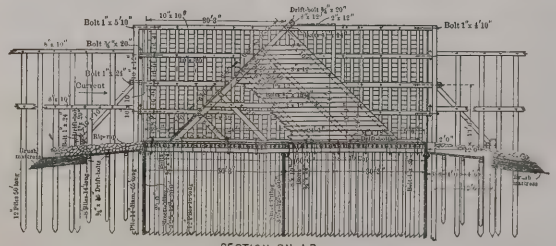
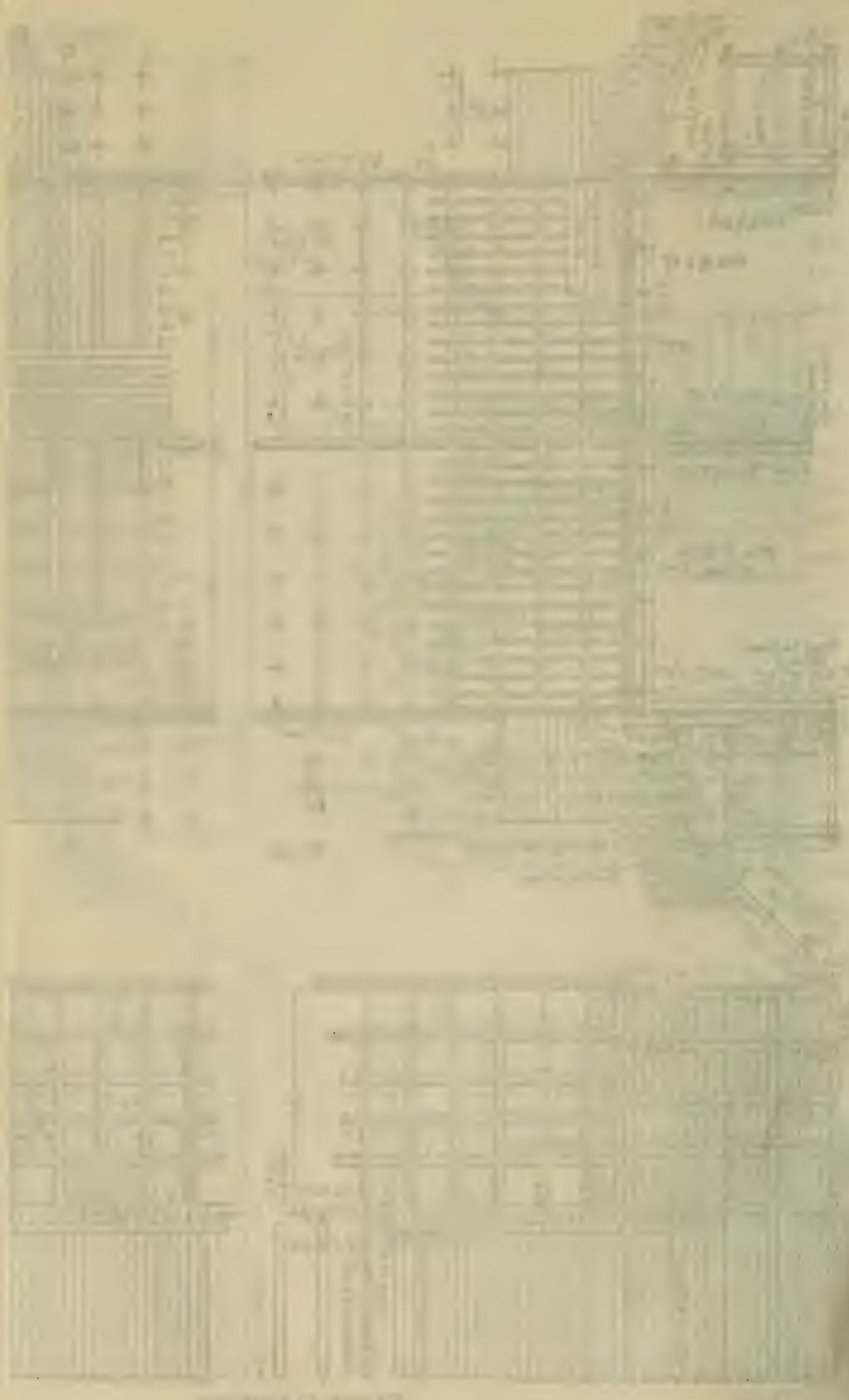




THE ROCKWOOD HEAD-GATE







resigned as Assistant General Manager and Chief Engineer, and was appointed Consulting Engineer, and the writer was appointed General Manager and Chief Engineer. Mr. Rockwood continued to act as Consulting Engineer until October 1st, 1906, when he severed his official connection with the company.

The San Francisco Fire.—On April 18th had occurred the earthquake which resulted in the great San Francisco conflagration, and exaggerated rumors as to the extent of the disaster made it seem certain that the machinery for the *Delta* was utterly destroyed; but that was the least important result, as far as the C. D. Co. was concerned. It appeared that the key city to the Harriman Lines was practically in ruins, and the Southern Pacific Company, as a railroad organization, was very seriously hurt.

Mr. Randolph hurried to San Francisco to join with the other officials in the West in conferring with Mr. Harriman, who had at once started for the scene. There, in the bustle and confusion of temporary offices, with the ruins of San Francisco still smoking, with the facilities of the road to carry people away from the stricken city taxed to the very utmost, with the wonderful railway system which constituted Mr. Harriman's life work crippled to an unknown extent, and with the financial demands resulting from the disaster impossible to determine, Mr. Randolph succeeded in inducing Mr. Harriman to advance an additional $250 000 for controlling the Colorado River and protecting Imperial Valley. It has always seemed to the writer that this was really the most remarkable thing in the whole chain of extraordinary happenings.

The Situation.—The wooden head-gate was completed, and the upper and lower by-passes connecting it with the break had been fairly well started with the dredges, *Alpha* and *Beta;* the concrete head-gate was well under way; the material for the hull of the *Delta* was in Yuma, and the machinery seriously damaged in San Francisco; the tracks of the Southern Pacific Railroad along the Salton Basin were nearly awash for a considerable length; the annual summer flood of 1906 had begun, and, from the Weather Bureau reports from the drainage basin, would be a very large one; the irrigation system of Imperial Valley was already threatened at several vital points by the excessive quantity of water going down the Alamo channel or Main

Canal; and friction between the old C. D. Co. stockholders and the new management had commenced.

No very great degree of reliance could be placed on the wooden head-gate, considering the character of its foundations; and the failure or serious weakness of that structure meant the failure and abandonment of the Rockwood plan for re-diversion. The difficulties of the Concrete Gate Plan, under the most favorable circumstances, became more apparent with further investigation, and were very greatly accentuated by the delay in getting the *Delta* into commission. The probability of the withdrawal of financial support at any time through the discouragement of the Southern Pacific officials as to the ultimate success of the work was a serious factor. Transportation facilities from Yuma were very inadequate, consisting of the steamers, *Searchlight, St. Vallier, Cochan,* and the barge, *Silas J. Lewis,* all of sufficiently light draft to navigate through the shoals and sand bars of the Colorado. There were large quantities of willow brush suitable for fascines and mattress work near the break, but no timber suitable for piling. The nearest point where piles and heavy timber were obtainable was Los Angeles; from there they came by rail to Yuma, from which point they could be floated down the river only at considerable risk, so that it was cheaper to load them on barges and bring them down with steamboats.

Experience thus far had indicated the practical impossibility of closing the break with a piling-brush-sand bag barrier dam, and there were no quarries for many miles either west or east along the railroad, and none, of course, available except with railroad facilities for loading and transportation. Further, rock would require to be transferred to barges at Yuma and be brought thence by river to the scene of operations.

Practically every engineer—and they included many of established national and international reputation—who had visited the break considered a rock fill barrier dam as entirely unworthy of consideration, for two reasons:

First, it was believed that rock would sink into the soft alluvial silt bottom and keep on going down indefinitely, even if more and more slowly. Old river men quoted numerous instances of wrecked river craft. They cited a dredge, bought a few years before by the C. D. Co., which had sunk on its way from Yuma to the upper intake,

gradually settling entirely out of sight in a few months. The consensus of opinion, therefore, was that any rock fill would certainly settle out of sight unless built on a very strong brush mattress foundation, and the probabilities were great that such a mattress would break under the load and fail of its purpose.

The second vital objection urged against a rock fill barrier dam was that the water going over it while building would dislodge some portion of the fill or some one rock at the top, thereby increasing the overpour at this point, which would dislodge more rock and in this way quickly result in a breach which could not be closed.

It was thought that these considerations not only quite precluded the idea of a barrier dam, should the wooden gate fail, but rendered very doubtful the construction of a diversion dam or obstruction in the channel opposite the gate which would cause a difference in head, above and below it, great enough to throw all the water through the by-pass and gate. This head was variously estimated at from 3 to 6 ft. —the head on the finished dam would be about 15 ft. at low-water stage.

On one point there seemed to be accord, namely, that the situation was a desperate one and without engineering parallel, and that there seemed to be little more than a fighting chance of controlling the river. No two, of the nearly fifty eminent engineers who visited the scene and examined into the situation more or less carefully, agreed on any one plan as offering the greatest chances of success, but pointed out fundamental weaknesses in practically all other methods suggested. This feature was so marked that when the writer suggested to President Randolph that the immensity of the interests dependent for their safety on the re-diversion of the river seemed to render advisable a Board of Engineers, he answered that he would regard 100 ft. of good strong brush mattress in place on the river's bottom as more valuable than the report of any Board of Engineers which could be gotten together.

The immediate menace, however, was from the summer flood in passing through the Imperial Valley to the Salton Sea. The Weather Bureau's reports from the upper drainage basin then indicated a very great total discharge, and a peak perhaps as high as 100 000 sec-ft. The crevasse had now enlarged, and the old channel below had filled up, so that practically all this water—several times as much as had ever yet entered the valley—must go the new way.

Summer Flood of 1906.—Plate XLVII shows that, compared with recent floods, the summer flood of 1906 was very large, although it has been greatly exceeded since then, notably in 1907 and 1909. The increased fall down the Alamo River channel resulted by August 1st in lowering the river at the diversion point approximately 4 ft., but it silted up as the flood receded, leaving a net lowering of between 2 and 3 ft. (Fig. 3.)

It widened the break from 600 almost to 2 700 ft., and rendered far more expensive, in time, equipment, and money, the task of putting the wooden head-gate into commission. The most important effect, however, was the danger it caused in various ways in the Imperial Valley proper.

Such a vast quantity of water going down the Alamo channel was, of course, never contemplated in designing the new waste-gates near Sharp's Heading discharging down the Alamo River (built June to August, 1905), and at Station 134 on the Central Main, and they were taxed to their absolute limits. So much passed the Alamo Waste-gate that it caused a recession of the grade in that channel below, so that the structure was, figuratively speaking, on stilts. Twice the chute below the structure had been extended, the last time in February and March, 1906, when the equipment was removed just as the water began to go over the top of the gates.

By a peculiar and most fortunate coincidence, when the Alamo Waste-gate was discharging approximately 3 500 sec-ft. and Sharp's and the Encina Head-gates were being utilized to the capacity of the canals below them, the water in the Alamo above this point spread overbank for miles, going to the west and south sufficiently deep to save the situation. Thus it happened that when the peak of the flood was reached, and approximately 75 000 sec-ft. were going down the Alamo channel toward the Salton Sea, all but about 5 000 sec-ft. were going overbank to the south and west. Had not this most fortunate condition existed, the Imperial Valley irrigation system would early have been broken into the deep channel of the Alamo below the waste-gate, and at once cut the water out of every canal.

Most of this overbank flow to the south and west collected in the various sloughs and low lands, particularly Beltran's and Garza's Sloughs, and flowed into the New River. The small channel of this watercourse was overtopped, of course, and the water spread out, just

south of the Boundary Line near Calexico, for a maximum width of about 10 miles. Some of the water overtopped the divide of the delta cone, gained the Paredones channel, and thence ultimately reached the Gulf.

The most critical points were where the New River channel crossed the Boundary Line, and a little farther down along the Central Main. At Calexico and Mexicali this broad sheet of water rose until it covered the ground about 4 ft. in depth. (Fig. 28.) The danger was not appreciated in time to throw levees to the west of the railroad track and thus protect that property. The disposition of the towns and the railroads was to wait for the C. D. Co. to build protective levees, in spite of that company's announced intention of doing nothing of the sort.* When the situation was finally realized, about 5 miles of levee—maximum height 5 ft.—encircling the two towns and connecting at the north and east with higher ground, was hurriedly built. Strong winds blow in the spring for two and three days at a time, and when such storms swept over a wide stretch, even though the ground had a considerable quantity of brush, waves were caused which made the maintenance of these levees at times very critical. They were held successfully, however, until the recession of the New River grade made them no longer necessary.

Along the Central Main, from near the branch railroad crossing west to beyond the "Five Gates" (where the canal turns to the north), the water rose so high during the last days of February that it overtopped the south bank of the canal (Fig. 29), and only by the most desperate work was it prevented from overtopping the north bank and sending water northeastward across the country to the Mesquite Lake Basin and the Alamo channel. Had this occurred, the Town of Imperial would have been most seriously threatened, perhaps destroyed, and the New River and Alamo chasms would have been joined by a third one, about 25 miles long, diagonally across the valley northeast and southwest. The C. D. Co. then greatly strengthened this north bank and raised it 4 ft. for a distance of nearly 3 miles. When the situation was most threatening the citizens of Calexico and Mexicali were called out to help hold the levees, while the people of Imperial rushed down to aid in the fight along the Central Main.

* This was because the company's attorney advised that it was not responsible legally for damages caused in the United States by operations of the Mexican Company in Mexico, and to avoid carefully any action which might be considered as an admission of responsibility by the company.

Both the Alamo and New River channels cut back, owing to the large quantity of water flowing in them, and the Salton Sea began to rise at the rate of approximately 7 in. per day. The Southern Pacific main line there was being shifted from time to time, by means of "shooflies." Along the branch line from Imperial Junction to Calexico the trouble at the crossing of the Alamo channel was far greater than should have been permitted. At no time was more than 3 500 sec-ft. going down the Alamo, yet this small quantity was permitted to eat away approximately 300 acres of land, in a semi-circular form, from the right bank of the channel where it is crossed by the branch railroad into the valley, and caused the railroad to "shoofly" its tracks five times. The alluvial soil of the Imperial Valley is very easily eroded, especially on the concave side of river bends, but it should have been possible to control at reasonable cost a stream of 3 500 sec-ft., with a velocity never exceeding 7 ft. per sec.

The Inter-California Railroad from Calexico toward Yuma had been constructed as far as Cocopah and practically all of this was under water. The Holtville Interurban Railroad, crossing the Alamo River, was cut out from time to time, the channel at that point being lowered more than 30 ft. This caused serious trouble with the discharge pipe of the Holton Power Company, the plant being left rather high, and considerable work was required to keep it from being undermined by side cutting. The head available, however, was increased by 30 ft.

When the grade of New River had receded to a point about 3 miles above the International Boundary Line, a large area of adobe formation was encountered, and the fingers of the stream began to eat away in various directions and threatened to tear up the country throughout that region in a frightful way. The rate of recession was also greatly slackened. Long before the peak of the flood had been reached, it was evident that the situation along the Central Main and at Calexico was very serious and must become much more so, until grade recession might give relief. It was decided, therefore, to use dynamite liberally in an endeavor to localize the New River's grade recession and to facilitate its progress.

From observations and soil and other data at the time available the probability seemed slight of such recession extending more than 6 or 7 miles beyond Calexico, or far enough to endanger lowering

FIG. 27.—ALAMO WASTE-GATE, NOVEMBER 17TH, 1906. ABOUT 30 FEET HEAD AGAINST GATE.

FIG. 28.—PORTION OF 4-MILE LEVEE PROTECTING CALEXICO AND MEXICALI, IN FLOOD OF JUNE, 1906.

FIG. 29.—OVERFLOW AGAINST WEST AND SOUTH BANKS OF MAIN CANAL NEAR FIVE GATES, 2½ MILES NORTHWEST OF CALEXICO.

the water surface in the Alamo above the controlling works and so menacing the water supply of the valley before the summer flood of 1908. It was known that very large areas of adobe soil existed in the Garza's and Beltran's Slough country, so that the cutting there would be very much slower. There thus seemed to be considerable leeway, while the strain on the irrigation system of the valley was so severe at several critical points that it was utter nonsense to think it could be held through another flood season.

In this dynamiting, from eight to sixteen ¼-lb. sticks of dynamite were tied in a bundle about a fulminating cap connected with from 8 to 12 in. of water-proof fuse. The fuse was then lit and the bundle tossed into the water. A little practice and careful observation enabled one to become quite proficient in estimating how far the bundle of dynamite would be washed down stream by the current before the cap exploded the charge, and in placing the charge to get maximum results. Undoubtedly, the course of the grade recession was considerably checked and bad erosion somewhat mitigated by this work, but it is very doubtful whether the time of the grade recession's passing Calexico and Mexicali was markedly accelerated.

When this occurred the results were spectacular in the extreme, the rate of cutting back at this point being fairly uniform at 1 ft. per min. The side cutting of the east bank of the wide, deep barranca for several days threatened Calexico, and carried away a considerable part of Mexicali, including the railroad station, brick hotel, and a number of smaller buildings. The actual damage sustained was about $15 000 in Calexico and $75 000 in Mexicali.

For a short distance past Mexicali the cutting back followed the borrow-pits of the Inter-California Railroad, utterly destroying it and carrying away much of the track and trestle material. About a mile out of town, the grade rose slightly above the flood-waters, but farther on, for several miles again, the roadbed was practically destroyed, although no more track material was lost.

These flood-waters covered about 6 000 acres of cultivated farms, of course, utterly ruining the crops. Greater damage, however, occurred as the grade receded and the water rushed from each side toward the newly-made channel, resulting in cutting back fingers or side cañons from the main stream to distances and depths depending on the length of time required to drain off the contributory water.

Some of these side cañons extended back from 2 000 to 2 500 ft. It thus happened that about 3 000 acres of improved and 10 000 acres of unimproved land were eroded to such an extent as to be practically ruined for agricultural or any other purposes. Of this area, about 7 000 acres were public land. The area occupied by the New and Alamo channels themselves was increased by about 7 000 acres.

The greatest damage in the Imperial Valley proper, however, was caused by the destruction of the flumes in the West Side Main over New River in Mexico and the Central Main over New River northwest of Imperial, leaving Mutual Water Companies 6 and 8 without water until January, 1908. These two districts contained about 30 000 acres in actual cultivation, and were rendered practically uninhabitable and absolutely waterless for about 1½ years.

Except as noted, agricultural operations in the valley were facilitated by the flood, there being at all times plenty of water in the canals. Prospective settlers, of course, were kept away almost entirely, but the inhabitants of the valley displayed a remarkable confidence that the trouble would be overcome, and business was not affected very seriously. Indeed, during these very times, the new and independent town site of El Centro was the scene of really wonderful building activity, and the Holton Power Company, directly and indirectly, practically doubled its plant and holdings in the valley.

The effect of this flood, in a geological way, was of extraordinary interest and very spectacular. In 9 months the runaway waters of the Colorado had eroded from the New and Alamo River channels and carried down into the Salton Sea a yardage almost four times as great as that of the entire Panama Canal. The combined length of the channels cut out was almost 43 miles, the average width being 1 000 ft., and the depth 50 ft. To this total of from 400 000 000 to 450 000 000 cu. yd. must be added almost 10% more for side cañons, surface land erosions, etc. Very rarely, if ever before, has it been possible to see a geological agency effect in a few months a change which usually requires centuries.

Preparation for Diversion Work.

All measures to prevent avoidable damage to the irrigation system in the valley from the flood-waters having been arranged, operations were resumed on the river. The break opposite the wooden head-gate

had been widened during the flood from 600 ft. to more than ½ mile, and necessitated work on a far larger scale than had ever been suggested. The opinions to the contrary notwithstanding, the ability to get rock in large quantities and rapidly seemed to the writer to be so essential, and it was so obvious that much better transportation facilities were required, that it was decided to build a branch railroad from the Southern Pacific main line at a point 7 miles west of Yuma (now known as Hanlon's Junction) to the break.

The located line of the Inter-California Railroad, construction of which had been stopped by the overflow waters at Cocopah, ran only a few hundred yards west of the wooden head-gate and 150 ft. west of the concrete head-gate. This Inter-California Railroad is a Mexican subsidiary of the Southern Pacific Company, and it was not difficult to arrange a change in its alignment to cross the Alamo where the best location for the diversion dam could be found and to build at once that portion from the break north to the concrete head-gate. Thence northward the permanent alignment was expensive and would require considerable time to construct, therefore it was decided to make a temporary connection of about 6 000 ft. from Hanlon's Junction to the concrete head-gate. It was arranged that the Southern Pacific should build the entire branch line and charge the total cost, on a force account basis, to the C. D. Co., and when later, if ever, the Inter-California Railroad should be completed, all that portion of the branch which could be incorporated with the permanent alignment of the road would be taken over by it at such a figure as it would cost at that time. The stretch from Hanlon's Junction to the western line of the lands of the C. D. Co. is in the Yuma Indian Reservation, and, according to the rules and regulations of the Interior Department, it would have taken some time to acquire a right of way for this portion. As it was feared that special permission might not be quickly obtainable, nothing whatever was said, but the line was simply built. Such a course was deemed justifiable, considering the gravity of the situation, the necessity for haste, and the very small discretionary powers given to Government officials in such cases. As soon as the existence of this track was no longer absolutely vital, permission was requested in the usual way and in due course was obtained. Construction of this branch line was begun on July 1st, and

on August 15th the first train load of materials passed over it to the Lower Heading.

Quarry.—The granite point of rock on which the concrete head-gate was founded seemed favorable for quickly developing a quarry where a large quantity of rock might be obtained, and instructions were given to do the best possible with it. The rock is a second-class granite, and, before the first closing was completed, a quarry had been developed with a 600-ft. face averaging 40 ft. in height. The development of this quarry and track room for outfit cars, locomotives, etc., called for the building of a large yard of sidings and spurs. This quarry was entirely on C. D. Co. land—that bought from Hall Hanlon at the very beginning.

Clay Pit.—Between the quarry and the Boundary Line, and about ¼ mile west of the branch railroad, there was an opportunity to develop rapidly a clay pit. Advantage was taken of this, and by the time the first closing was completed, there was a steam shovel face 600 ft. long and averaging 60 ft. in height. The clay in this bed is rather hard and requires some blasting, but it melts down in water, and when mixed in about equal proportions with the cement gravel from the Mammoth gravel pit makes a very impervious material for dam construction.

The Mammoth Gravel Pit.—This pit is on the Southern Pacific Railroad 41.08 miles west of Hanlon's Junction. It had been thoroughly developed at that time and had been used for ballasting the main line for more than 100 miles in each direction. It is the property of the railroad, and the material obtained there is fairly high in clay, the result being essentially a cementing gravel, which makes the surface of the track almost impervious.

Other Quarries Available.—At Declez, a point on the Southern Pacific Railroad 195 miles west of Hanlon's Junction and 49 miles east of Los Angeles, there is a large, well-equipped quarry of very good granite, from which material for the construction of the breakwater at San Pedro Harbor, 19 miles southwest of Los Angeles, is obtained. The output of this quarry is very large, the rock running up to 12 tons.

Near Ogilby, 7 miles west, a large area is covered with lava "nigger-head" rock, essentially one- or two-man size, which had been in part denuded to furnish rip-rap around the railroad bridge over the Colorado

at Yuma. The tracks, however, had been torn up, and no stone had been obtained therefrom in years.

At Tacna, 52 miles east of Hanlon's Junction, there was a quarry formerly used by the railroad but abandoned because the rock therefrom was small and of poor quality.

At Patagonia, on the branch line south from Benson toward Nogales, and 370 miles east of Hanlon's Junction, there was a well-equipped quarry controlled by the Southern Pacific. Its output was a reddish limestone, considerably smaller than that at Declez, but yet frequently turning out 10-ton rock.

These four sources of supply constituted the utmost possibilities, aside from the quarry which might be developed at Andrade.*

Brush.—By no means all the area contiguous to the Colorado is covered with willow brush, but it occurs in spots, often of very large extent. Such areas on the west bank of the river near the Edinger Dam had been cleared away, and west of the old Main Canal there was an old shallow lake which, though now drained, was practically barren. All brush, therefore, had to be obtained from the south side of the break, and with an average wagon haul of about 1 mile. The growths, ranging from 6 to 18 ft. in height, were ideal for fascines and mattress work. Main and branch roads were cut by Indian labor in order to get this material to the front rapidly.

Dredges.—The dipper dredge, *Alpha,* and the suction dredge, *Beta,* were in reasonably good condition, but the former could not be used in the sand bar left exposed in the bottom of the break when the waters receded, because the material slipped down to such a flat slope that it would have imprisoned the craft. After doing all it could in the by-pass and more solid ground, it was started to deepening the old Main Canal toward Algodones. Dams were built behind it from time to time, and water was pumped into the canal at the upper intake to keep the machine afloat. The quantity of water required indicated a surprisingly small seepage loss from this old canal into the surrounding country, and this is in accord with the unexpected experience with the coffer-dams of the wooden and concrete head-gates.

Steamers and Barges.—During the latter part of 1905 the Mexican Co. purchased the steamer, *Searchlight,* 91 ft. long, 18 ft. wide, and

* Andrade is the name of the Inter-California railroad station on the American side of the Boundary Line, Algodones being on the Mexican side.

drawing, without load, 18 in. of water. It had a barge, about 55 ft. long and 25 ft. wide, on which most of its load was carried. The steamer, *Cochan*, 135 ft. long and 31 ft. wide, the largest on the river, belonged to Yuma parties, and as it had been leased by J. G. White and Company for hauling materials and supplies to the Laguna Weir, it was not available. There was another steamer on the river, the *St. Valliers*, 75 ft. long, which was a little smaller than the *Searchlight* and in very poor condition. In addition to these there was the barge *Silas J. Lewis*, 115 ft. long and 35 ft. wide, which was fitted with a donkey engine with which it pulled itself up stream. This barge was rented for $15 per day, and its deck was cleared for mattress weaving.

Grading Outfits.—The Southern District of the Southern Pacific Company—from Santa Barbara and Fresno, Cal., to El Paso, Tex.—has enough reconstruction and betterment work to keep two or three grading contractors' outfits at work except during the very hot season. An arrangement was made with one of these, Shattuck and Desmond, to supply an outfit on the force account schedule paid by the railroad, with provisions for the payment of all duties and for all stock dying from heat. This firm secured, fed, and housed its own laborers. Inasmuch as there was no very definite plan as to the work which would be required, no contracts were feasible, hence the force account arrangement. At one time about 800 head of this firm's stock, with complete camp equipment, Fresno scrapers, plows, etc., were on the work.

Materials and Stores.—Arrangements were made with the Southern Pacific for equipment, materials, and stores, on the basis of cost plus 10%, and for freight charges of 0.5 cent per ton-mile, until the provisions of the Interstate Commerce Commission prohibiting such freight arrangement went into effect. Two steam shovels were brought in for quarry work and one for the clay pit. Complete work trains were requisitioned from time to time until a maximum of ten was reached. A roundhouse foreman and an assistant master car repairer were sent by the railroad company, and temporary, but effective, plants were installed at Andrade. Three carloads of repair parts and stores for engine, car and air-brake repairs were sent out, used from, and returned when the work ended. All requisition blanks, rules, and other organization methods of the railroad were continued.

When the Southern Pacific built the Lucin Cut-off, consisting of a long trestle bridge and an immense fill across Great Salt Lake, in Utah, there were bought a large number of steel side-dump cars, of 45 cu. yd. capacity, locally known as "battleships," weighing approximately 20 tons, and having a capacity of 100 000 lb. with a permissible 10% overload. These cars were frequently loaded to 125 000 lb. on this work, as the trip between the Andrade quarry and the break did not exceed 4 miles. At first 80 of these cars were secured, and more and more were sent until about 300 were finally in service. Such a quantity of railroad equipment necessitated rather extensive terminal facilities, and these were provided on the American side of the line because of the customs regulations of the Mexican Government.

The railroad from Hanlon's Junction to the Lower Heading, the quarry, clay pit, steam shovels, etc., were under Mr. Eulogio Carrillo, Assistant Engineer of the Southern Pacific Construction Department, from June 1st, 1906, to July 21st, 1907, as a superintendent of the C. D. Co., from which he received his salary, the railroad giving him leave of absence for that period. All the men under his direction, however, were carried on the Southern Pacific payrolls, and bills were rendered later by that corporation to cover this expenditure.

There were two reasons for having the railroad company supply so great a quantity of labor, equipment, materials, and supplies. First, it afforded an opportunity to assemble quickly a thoroughly organized and efficient force of men; the advantage of obtaining materials and supplies at low prices by the purchasing department of the Harriman systems; immediate shipment of repair parts not kept on hand, thus reducing delays to the minimum; and the ability to increase or decrease rapidly the force and equipment without confusion. The second reason was that no immediate cash was required, and as bills of all kinds were not usually presented and approved in less than about 6 months, approximately 3% in interest was saved. All bills were rendered at actual cost plus 10%, which thus meant really cost plus 7%—a very low figure for superintendence, etc.

Whenever any train, equipment, or men left the main line and came on the branch line they reported to and were under the jurisdiction of Mr. Carrillo, who in turn reported to and was under the sole jurisdiction of the writer. In this way no misunderstanding arose, and the entire force obeyed instructions as quickly and fully as though

there were absolutely no connection between them and the Southern Pacific.

Storehouse at Lower Heading.—Duty had to be paid on everything taken into Mexico, but, nevertheless, a very complete storehouse of repair parts, small tools, etc., was established at the Lower Heading. No requisition system was put in, however, because it was felt that the losses which would thus occur would amount to much less than the delay due to any form of red tape, whatsoever. Everything received was charged to the work, and at its closing down an inventory was made and the work was credited with the value of the material left.

Climatic Conditions.—From about June 1st to the middle of September or October 1st, the temperature of this region is so high that until 10 years ago it was not considered advisable to continue large construction work during that season. There can be no doubt that ordinary labor is only from one-third to two-thirds as efficient in such heat, and during this particular year the general average seemed to be about one-half. There is little wind during this period, and the humidity is ordinarily very low, though occasionally it is quite high for periods of two or three days.

Mosquitoes are frequently a terrible pest, very often driving even cattle out of regions near stagnant water. There is relatively little vegetation about Andrade, and at the Lower Heading a large camp compound was entirely cleared and the stagnant pools in the vicinity drained at a slight cost, so that the mosquitoes, while annoying, were by no means serious.

Brush and arrow weed growths are so dense that white men, no matter how well acclimated, cannot work very hard in cutting them down. Men from the central part of Mexico were imported, but they could stand it little better. Indian labor is the only kind for that sort of work.

Labor Conditions.—The work of rehabilitating San Francisco after its disastrous conflagration drew there an immense amount of shifting labor. To the south Los Angeles was growing in every direction. The Harriman Lines, under President Randolph, was employing large numbers of men constructing the West Coast Railroad from Guaymas toward Mazatlan and Guadalajara. Much betterment work was in progress on the lines from Los Angeles to El Paso, and large forces were required for building "shooflies" and shifting track along the

Salton Sea. J. G. White and Company was rushing work on the Laguna Weir, and the Reclamation Service was building the Roosevelt Dam near Phœnix. Thus the labor situation in California as a whole, and in this part of California in particular, was acute. The immigration laws of the United States prevented the importation of Mexicans, except in a very small way, but here the work was in Mexico. It was decided, therefore, to obtain laborers from Central Mexico, ship them from El Paso to Yuma in bond, and back into Mexico at the Lower Heading. Arrangements were made with the Labor Agent for the Southern Pacific, Southern District, Mr. Ben Heney, of Tucson, to ship 500 men. This plan was an utter failure, for two reasons. The Mexican officials did their best to prevent Mr. Heney's agents from getting men started, and the 75 men who arrived were unable to stand the climate.

Attention was then turned toward getting Indians in large numbers, and arrangements were made with Mr. C. E. Dagenette, United States Indian Outing Agent, with the result that, by the time work was in full swing, practically all the men, women, and children of six Indian tribes were on the work—the Pimas, Papagoes, Maricopas, and Yumas, from Arizona; and the Cocopahs and Diegueños, from Mexico. These six tribes fraternized and got along together without any difficulties whatever, and constituted a separate camp of about 2 000 people. About 400 workmen could be depended on from this collection. They were paid 20 cents an hour, and every 9 men received in addition one man's pay to go to a squaw for cooking their food. The Indians bought their own supplies, and to avoid duty built their camps on the Arizona bank, crossing the dry channel below the break to and from work.

Indian labor was very satisfactory, and, indeed, just what other arrangement could have been made is very problematical. Under intelligent foremen who understand their peculiarities, chief of which is lack of assurance and consequent timidity in going ahead with work, they are quite satisfactory. They must be paid weekly, and very few can ever be induced to work on Sunday or to put in overtime, regardless of how critical the stage of work may be when the whistle blows.

Very fortunately, indeed, an unexpectedly large amount of floating labor came in from every part of the United States, men who are attracted to any work which has achieved notoriety for any reason.

Once on the ground these men did not work any great length of time. A work train ran into Yuma every night for provisions and supplies, returning early in the morning, and it always carried a considerable number of cheerful capitalists out and sadder and wiser men in. Yuma at that time was "wide open," with all sorts of lures which few of these floaters could resist. To what extent the work would have suffered had Yuma then been a closed town, as it is now, is a question.

The general wages paid were:

Pile-driver foreman .....50 cents per hour.
Pile-driver donkey runner.....43¾ " " "
Good pile-driver helpers.....31¼ to 37½ " " "
Ordinary labor.....27½ to 30 " " "
Work from 8 to 10 hours per day.
Board deduction, $22.50 per month.

Commissary and Camp Plans.—The usual outfit cars were provided for all men carried on the rolls of the railroad, and many were boarded in the dining cars, which were a part of Mr. Carrillo's permanent construction outfit. The remainder of the men were boarded by Mr. M. C. Threlkeld, of San Francisco, who had and still has a contract with the railroad to board all gangs engaged in maintenance of way and betterment work on its lines. Mr. Threlkeld took an essentially similar contract for feeding the white laborers of the C. D. Co., the first contract being for 25 cents per meal in the United States and 40 cents in Mexico, the contractor to pay all customs duties on material and supplies. After the second break, and when the work was continued at President Roosevelt's request, it was deemed probable that the Mexican Government would refund duties on provisions thereafter, so that the contract was changed on January 1st, 1907, to 25 cents per meal, the Mexican Co. to pay the duties. This contract covered meals for all white laborers, including men on dredges, on the steamer *Searchlight,* etc., and gave Mr. Threlkeld the exclusive selling of clothing, tobacco, notions, etc., to the laborers. The Indians bought relatively little from him, however, preferring to deal with Yuma merchants with whom the local Indians were very well acquainted.

Excellent board for the men was insisted on and furnished. It was believed that good board, especially with lots of fresh vegetables, would be a large factor in keeping men on the work, and this was found to

be the case. Large numbers of mosquitoes were feared, in spite of precautions taken, so bunk houses were built, with brush ramada roofs, and carefully and effectively screened all round. These precautions were not exactly necessary, but were nevertheless well worth their cost.

Policing of Camps.—The many different classes of laborers on the same job and under Mexican laws made it essential to have effective police arrangements, and bar liquor from the camp absolutely. The Yuma Indian Reservation extends to the line, and, in addition was then and until 1908, a part of San Diego County, and a "dry" region. Across the river in Arizona is "wet," but the United States laws against selling liquor to Indians are rigorously enforced. In Lower California, however, the idea of liquor control has not even germinated, and it was necessary to promise to prevent American Indians from getting liquor in Mexico before permission could be obtained to take them out of the United States—and this was quite proper. Accordingly, arrangements were made with the Mexican authorities to put the entire region under martial law, and send a force of rurales with a military commandant at their head to police the camps. This proved extremely efficient and satisfactory, and there was absolutely no disorder at any time.

Customs and Duties.—Except for the operations of the C. D. Co., there was no development in Mexico along the river; therefore, until 1908, the nearest custom house in Lower California was at Mexicali. A garrita was maintained at Algodones, however, where material going down the river to land in Mexico was passed. During the construction of the Edinger Dam, all camps and supplies were kept on Disaster Island in the middle of the river, so that there were no customs charges. When the construction of the wooden head-gate was begun, endeavors were made to get the Mexican Government to establish a customs office at Algodones temporarily, but without success. Accordingly, all bills of material to be passed had to be sent to the custom house in Mexicali; there the charges were assessed, and the manifest was returned to Algodones before the goods could be taken over, which was very cumbersome and slow.

Another method of getting goods across the line was taken advantage of, namely, by boletas. The Mexican Government permits each individual, on payment of duties, daily to take across $20 (Mexican) worth of dutiable stuff without manifest, and the authorities agreed to

permit goods to be passed at the Algodones garrita by this boleta method, having individual employees of the company sign the boletas. In this way emergency stuff was passed.

Under the concession of the Mexican Co., machinery and materials for permanent construction was to be admitted without duty, but the intention of this provision was plainly for the company to make out a list of what would be required once for all, and that such freedom from duties would apply to the original entry of the machinery and material, and not to subsequent repair parts, etc. Obviously, it did not contemplate the refund of customs charges in such a case as closing the crevasse. Nevertheless, it seemed probable that the customs charges for material and supplies other than provisions would be refunded, because the Mexican Government itself was vitally interested in stopping the break. Tentative negotiations toward this end were started, but the procedure for securing such permission is a long one, and it was advised that the work be prosecuted and the request for refund made after its completion. It was also made plain that no refund would be given for duties on provisions, as it was impossible to determine that the provisions passed were all actually used on the work. When the work was completed a request for a refund was made, and, on President Diaz's recommendation, the National Congress, by vote, refunded approximately 75% of all duties paid, amounting to more than $40 000.

The chief objection, therefore, was the red tape involved in passing goods, and the delays which followed any slight technical mistake in classification. As an illustration: an inspector investigated the customs transactions of the period about a year later, and assessed a fine against the company for $3 000 for utilizing the boleta method of passing emergency materials and supplies. On proper presentation of the facts, however, this fine was remitted. Stock with harness and grading equipment was permitted to be passed into Mexico under bond for a period of 6 months, as also was machinery, which provision assisted very greatly in the work.

All payrolls, time checks, receipts, and legal papers require stamps to be affixed and cancelled, inspectors from time to time visiting all corporations and checking the books. If any irregularities are found in the books or papers for the 6 months immediately preceding, such inspector is then permitted to go back to the period of 6 months imme-

diately preceding that, etc. If, however, everything is regular for the first 6 months preceding, that operates to prohibit inspection prior to that time. These inspectors get a considerable percentage of fines assessed and collected, and are consequently quite zealous, so that it is profitable to obey the stamp law scrupulously.

Necessity for Mexican Corporation Doing Work.—On taking charge of the affairs of the Mexican Co., the writer found that up to that time work done in Mexico had been paid for on the American side of the line through the C. D. Co., and in this way no Mexican stamps were required for payrolls, time checks, etc. In other words, the C. D. Co. had its forces go over into Mexico and do work on the canals of the Mexican Co. directly. As this was obviously contrary to the spirit of the Mexican laws on the subject, arrangements were made at once whereby the Mexican Co. did all work in Mexico and billed the C. D. Co. therefor at actual cost, the C. D. Co. turning over all materials and supplies required on the Mexican side of the Line at its expense.

Mr. A. F. Andrade, now Depositario for the Mexican Co., and Assistant General Manager of the Inter-California, was made General Agent of the Mexican Co., and was in charge of all negotiations between that corporation and the Mexican Government, and to his tact, energy, and ability is attributed the relatively small amount of irritation and delay encountered.

Occasionally, rules and regulations had to be disregarded, and this was done when it was deemed quite necessary, knowing that the local officers would report such infractions of the laws, but that the higher officials would view such infractions very sensibly when sooner or later brought to their notice with full explanations. For example, before permission was given to run trains into and out of Mexico after dark, a serious situation developed just at sundown, immediately requiring rock at the Lower Heading, and the Mexican officials at the Boundary Line would not permit trains to pass. Their protests were disregarded, for while the officials under the circumstances could not act otherwise, it would have been folly not to have disregarded their orders, considering the urgency of the matter. Proper explanations were at once made, and the company was not criticized in any way for the action.

Difficulties in doing work in Mexico are largely due to ignorance of Mexican conditions, customs laws, and personal characteristics, and doubtless are no greater than a Mexican would encounter in doing work in the United States. It is very desirable for the highest officer in charge of work to speak Spanish well, as minor Mexican officials are far more impressed with a statement coming from him than from any subordinate officer.

Methods of Diversion of River Through Rockwood Head-Gate.

The triangular space between the two faces of the **A**-frame and the horizontal cross-bracing of the wooden head-gate was made into a long pyramid, by flooring the bottom and sides, which was filled with sand taken in by wheel-barrows, in order to give additional weight to the gate in resisting the buoyant effect of the water.

By August 5th the discharge of the river had fallen to 24 500 sec-ft., and directly beside the Rockwood Head-gate the receding waters had exposed sand bars on each side of the main channel—the situation being as represented by Fig. 23. When these sand bars had dried sufficiently, teams were used in throwing up an embankment on the line of the diversion dam. Brush jetties were also used to narrow the channel, the *Beta* assisting. In a little more than a week the stream was narrowed to 600 ft., the river gradually falling. Work was then begun on weaving a brush mattress, 100 ft. wide up and down stream, and sinking it on the bottom of the river. The decks of the barge, *Silas J. Lewis,* were cleared and skids were rigged thereon; ½-in. steel cables, 8 ft. apart, were anchored to "dead men" in the north bank and unwound from spools beneath the skids, such cables constituting the longitudinal strength of the mattress; and to these were fastened brush fascines averaging 18 in. in diameter and 100 ft. in length. These fascines were built up between vertical pins at the upper end of the skids and bound with baling wire, and as they were completed they were pushed down to the last one in the mattress and sewed to it and to the supporting cables with ⅜-in., 9-strand, galvanized-iron cable and cable clamps. Fig. 30 shows the method of sewing and fastening. When a length of mattress equal to the width of the barge was completed, the barge was slowly pulled from under it, and it caught the silt and at once settled heavily to the bottom. No kind of weighting whatsoever was required. Another

FIG. 30.—WEAVING BRUSH MATTRESS.

FIG. 31.—DOWN-STREAM END OF BRUSH MATTRESS ABOVE WATER BECAUSE OF SILTING ACTION.

barge width of mattress was then woven and sunk, and so on. Figs. 30 and 31 show the method of constructing the mattress and the number of men employed.

It required 20 working days, with two shifts, to weave and sink two mattresses, one on top of the other, across the bed of the stream, or a total of 1 300 ft.; thus the average rate was 65 lin. ft. or 6 500 sq. ft. daily. The work went ahead without interruption or difficulty except that once the anchor lines controlling the barge were not handled with sufficient care and the first layer of mattress was not sunk across in a straight line, but curved down stream in the middle perhaps 20 ft. at the maximum point. This, however, was not important.

The prevailing idea as to the necessity for such bottom protection in the river may be better realized from the fact that several engineers with the longest experience on the river joined in urging that a solid canvas back be sewed on the under side of the mattress. It was feared that the water might start a wash through a break in the mattress, that such a stream would carry the sand from below, cause a depression for the mattress to span, and result in breaking it when weight should be put on above. This, however, was deemed unnecessary.

While the mattress work was being completed, a 4-pile railroad trestle with 10-ft. bents was started across the center line of this foundation, decked, and a railroad track built thereon. This trestle was driven from both ends, and was ready for the passage of trains on September 14th, 6 days after the completion of the mattress. In the mean time, the earthwork across the north sand bar had progressed sufficiently to connect the rails, so that trains could run out on the trestle. On the south side, the jetty work and the *Beta* had built up a sand bar on which a frame trestle on mud-sills was erected, connecting the earth embankment on the south sand bar to the trestle, thus affording tail room for trains. This frame trestle was filled in with material from the clay pit at Andrade.

At this stage, brush fascines were put in between the bents of the trestle over the channel, laid longitudinally with the stream, and sunk by rock from the quarry at Andrade. The rock was loaded into "battleships" with a steam shovel, hauled down, and dumped from the trestle. In this way a difference of 6 ft. in water elevation above and below this diversion dam was attained with no difficulty whatsoever.

Meanwhile the by-pass in which the Rockwood head-gate stood was being enlarged in several ways. The *Alpha* had cut a small channel from the crevasse to the gate from above and from below, through the solid ground, and the *Beta* had enlarged these cuts until it was taken over to assist in the jetty work on the south side of the river. A small ditch was cut with teams and scrapers across the sand bar, as an extension of the down-stream end of the by-pass. This channel was excavated to the water-table with Fresno scrapers, and made as narrow as possible, reliance being placed on enlarging it by the erosion of the water. In two or three places adobe deposits of considerable extent were found, and in these dynamite was used, as already explained.

The steamer, *Searchlight,* was anchored in the upper by-pass for two or three days with its rear end against the bank and the stern wheel kept going as fast as possible. This greatly hurried the erosion. The increasing head on the diversion dam aided these methods of enlarging the capacity of the by-pass until on October 10th only about 1 450 sec-ft. of the river's total discharge of 14 300 sec-ft. was not going through the gate.

The alignment of the by-pass was unfortunate, as it had quite a sharp curve, and the upper end left the river at a sharp angle. At this point cutting began, and to prevent it a small brush mattress was woven and weighted down with rock.

It was soon seen that, with the 4-ft. openings between them, the **A**-frames of the gate caught the drift in the water very badly. Anticipating this, cables had been stretched across the entrance of the by-pass and fitted with grab-hooks, like fish hooks on a trout line. These grab-hooks were of $\frac{3}{8}$-in. wrought iron fastened with from 6- to 8-ft. lengths of sewing cable to the cable spans at intervals of about 8 ft. It was hoped that these would catch drift where it could easily be removed, and prevent trouble at the gate. However, they did very little good.

When the current through the gate increased to 6 or 8 ft. per sec., a scour developed both above and below. Soundings showed that the scour below the gate was not at all serious, but was really far less than had been anticipated. The eddies at the ends of the gate caused side-cutting, as is always the case, but really nothing alarming. The scour above the gate, however, was surprisingly great; some was expected, but not nearly as much as occurred. Brush and

rock extension of the apron, as shown on the plans, had not been put in as it had been the intention to use rock from Andrade in lieu thereof. When soundings, which were taken frequently, showed that the by-pass bed was eroded to the level of the floor of the gate, approximately 1 000 cu. yd. of rock were loaded on a barge which was swung in front of the gate and held by cables until unloaded.

Failure of Wooden Head-Gate.

On October 3d a serious settlement of the earth filling in the north abutment suddenly occurred. Excavation was at once made to ascertain the cause, and some small leaks in the end wall on the up-stream side of the **A**-frame were found. These were stopped up, and the earth was leveled to only a few feet above the water surface on the outside. Two days later the lower wing-wall in this same abutment spread out at the bottom on the west side. The gate itself buckled up about 0.3 ft., about one-third of its length from the abutment, such buckling apparently occurring very slowly within 24 hours, ending on October 5th. These signs of weakness were accompanied by the tearing up of the up-stream apron in relatively small sections, which were at once thrown against the **A**-frames by the current. With great difficulty these were taken out piecemeal, and then only in part. These, together with the drift which accumulated, caused a head of 4.4 ft. on the gate on October 11th. At this time the discharge through the gate was about 12 000 sec-ft.; the maximum discharge through it was about 13 000 sec-ft. on October 8th.

These indications of weakness showed that it would not be safe to use the gate after closing the break, and that it would be very fortunate if it held until this could be accomplished. Furthermore, the drift made it very difficult, if not impossible, to set the flash-boards. Accordingly, on October 5th, a pile bridge was begun just above the gate and connected with the track to the south by a frame bent trestle supported on mud-sills—the same construction as had been utilized on the south side of the channel. This trestle was finished in the morning of October 11th, and it was intended to dump rock from it and fill up the gate in this way and not attempt to use the flash-boards.

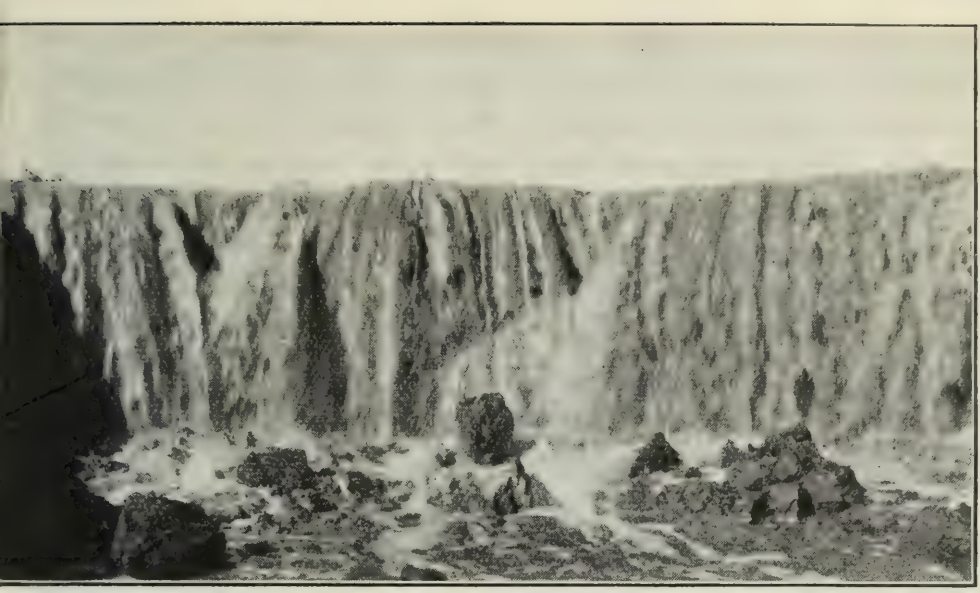
When the first rock train was slowly pushed over the trestle, at 11 A. M., three bents of the frame trestle settled and wrecked the train,

fortunately injuring no one seriously. Just why construction, which on apparently the worst ground on the south side of the main channel was entirely satisfactory, should have failed here, is not known—things happened thereafter too rapidly to find out. At any rate, had the trestle stood and had the large number of loaded "battleships" held ready been dumped, the writer has always believed that the head-gate would not have failed utterly. Be that as it may, at 2:30 P. M., without any warning, the gate suddenly buckled up at a point about one-third of the way from the south abutment (Fig. 46), and the larger portion—from there to the other abutment—floated down stream about 200 ft., where it lodged. The remainder of the gate stayed in place, although it settled in the central end. When the gate went out, the 4.4-ft. head above it caused a destructive wave of water, carrying large quantities of drift and débris from the wrecked gate against the railroad trestle crossing the by-pass about 300 ft. below. In about 5 min. this damaged the trestle seriously, and would have marooned a locomotive and train standing on the south side of the by-pass had not the engineer taken chances and pulled across before the piling began to go out.

The pond above the diversion dam extended some distance up stream and contained a large quantity of water which had to run out before the flow through the by-pass was reduced to the discharge of the river. By the time this occurred, considerable inroad had been made at the point where the upper by-pass left the river—which had been protected by a small brush mattress—and for a time it threatened to work down to and through the earth portion of the dam. Aggressive work was centered there, and such action was finally arrested.

Closing the Break with Rock Fill Barrier Dams in Series.

The lowering of the water above the diversion dam left it dry, except for a surprisingly small quantity of leakage, and enabled examination of the rock fill which had been produced by an ever-increasing proportion of rock with respect to brush. This condition of affairs seemed to indicate that the reasons urged why a rock fill dam of considerable height could not be built in a running stream were not altogether strong, and suggested the possibility of very quickly controlling the situation with a series of rock fill dams, each of which should sustain a head of not more than 4 ft. This particular dam



IG. 32.—Grade Recession in New River Near Calexico. Maximum Rate of Recession, 1 Foot Per Minute. Drop, 28 Feet. June, 1906.

Fig. 33.—The Hind Dam, Passing 7 000 Second-Feet. Head, 10 Feet.

had stood successfully a head of 6 ft. without any of the troubles prophesied for constructing rock fill dams in streams. Furthermore, the tracks of the Southern Pacific Company on the Salton Sea were in an extremely critical condition, and the southern transcontinental line would soon be interrupted, at an estimated cost of $1 000 000 a month. It was obvious that, if this were to be prevented, very quick action was necessary, and if hope should be abandoned, withdrawal of financial support in controlling the river was almost a certainty. Furthermore, other plans of controlling the situation possessed most serious difficulties, as already explained.

As a matter of insurance, however, a rush order was wired for additional sewing cable for building a diversion dam across the Colorado directly opposite the concrete head-gate, exactly as had been done successfully opposite the wooden head-gate to divert the river through the former structure, and as had been done with the other, trusting to dynamiting, dredging, erosion, etc., for enlarging the 4 miles of Main Canal thence to the break. This done, the trestles across the by-pass, above and below where the wooden head-gate had been, were repaired, and a third trestle, 30 ft. above the lower one, was hurriedly thrown across this stream, which was carrying the entire flow of the river, the waterway through the opening of the gate being only 120 ft. wide.

Such method of closing the break and forcing the river down the old channel by three rock fill barrier dams in series was therefore considered problematical only because there was no mattress under any of them, and the brush mattress idea had always been regarded as essential. The branch railroad from Hanlon's Junction to the Lower Heading was now in excellent condition, and the Andrade quarry was sufficiently developed to permit the use of the two steam shovels, producing about 5 000 cu. yd. of rock daily, by working night and day. It was felt that with these facilities, together with the rock which could be obtained from quarries within a distance of 400 miles to the east and west, rock could be put into the stream faster than the water could carry it away.

As a matter of fact, these three dams were built up so rapidly and successfully that only 10% of the water was going through the by-pass by October 29th, most of the remainder—8 600 sec-ft.—going over the diversion dam with the mattress foundation. Here, a sec-

ondary trestle, with 4-pile bents, 15 ft. apart, parallel to and 30 ft. up stream from the first, had been rushed, and from the two a rock fill dam was completed, turning all the water—9 270 sec-ft.—down the old channel and actually closing the break on November 4th. That is, after working on other lines continuously for 15 months, the stream was controlled by a rock fill dam in 24 days. In other words, the rock fill barrier dam plan, which had not been advocated, or indeed seriously considered, by a single man, proved to be a very simple and efficient, though expensive, method of re-diverting the river. The fact that there was a very substantial brush mattress foundation, however, was deemed by many as of vital importance.

Leakage through the structure was stopped by dumping "battleship" trainloads of gravel from the Mammoth gravel pit and clay from the clay pit, the whole being puddled with fire streams. The *Beta,* which was kept above the diversion dam in order to be taken up the river and used in the intake above the concrete gate, was used in widening the up-stream toe of the dams.

A week and a half after the failure of the wooden head-gate, the success of the series rock fill dam plan seemed assured. The *Alpha* had finished its trip up the Main Canal and cut into the excavation in which the concrete head-gate had been built. The intake from the river to the concrete head-gate was completed, and by October 29th the river at this point had been raised approximately 4 ft. by operations at the break. The dam holding out the river here, and those which had been left by the *Alpha* on its way up, were blown out, and water commenced to flow through the concrete head-gate and Main Canal into the Alamo channel below the diversion operations. The initial discharge was about 150 sec-ft., and had increased but little when the river re-diversion was complete. At that time (November 4th) the water height at various points was as follows (C. D. Co. datum):

Above the dam	113.0 ft.
Below the dam	97.3 "
Opposite concrete head-gate	114.5 "
Floor of concrete gate	98.0 "

By November 15th only 300 sec-ft. were flowing in the Main Canal, the fall of 17 ft. in these 4 miles not having resulted in much erosion, because of several stretches of adobe deposits, though the current was

FIG. 34.—SEALING THE HIND DAM WITH GRAVEL AND CLAY BY HYDRAULIC JETS, NOVEMBER, 1906. NOTE SLOPE OF DOWN-STREAM SIDE OF ROCK FILL DAM.

FIG. 35.—GENERAL VIEW OF THE SECOND BREAK, JANUARY 20TH, 1907.

quite strong. Dynamite was used liberally, and by December 5th the grade recession was within 1 mile of the head-gate. In this way continuity of supply into the valley was kept up, and the water users suffered relatively little inconvenience.

In making the first closing, rock was unloaded from the three trestles across the by-pass and two trestles over the main channel. Records were kept daily of car loads of rock from Andrade and from the distant quarries unloaded from each trestle, but this record, unfortunately, has been misplaced, and the totals obviously signify nothing. As the quantities of various materials used during the entire period from August 1st to November 4th may be of interest, they are given in Table 13.

Completing the Hind Dam.

The dams across the break and the by-pass were hurried to completion with material from the Mammoth gravel pit and the clay pit at Andrade. It was decided that the structure should have a top elevation of 124, and that meant increasing its height fully 8 ft. The tracks over the trestles were raised so rapidly that no attempt was made to recover the stringers or caps.

TABLE 13.—Approximate Data of Constructing Diversion Work on Colorado River.

2 200 cords of brush and 40 miles of steel cable used in mattresses and shore protection.
3 800 ft. of railway trestle.
15 200 ft. of 8 by 17-in. Oregon pine stringers.
1 100 piles.
1 690 cars (50 000 cu. yd.) of rock (90% used from October 11th to November 4th).
841 cars (32 000 cu. yd.) of gravel.
808 cars (33 000 cu. yd.) of clay.
200 000 cu. yd. of earth, placed by teams.
200 000 cu. yd. of earth, placed by dredges.
200 to 500 head of mules and horses working from July to November 20th.
200 men in June, increasing to 1 000 men on November 4th.
Discharge of river, June 27th, 99 200 sec-ft.
Discharge of river when actual work of constructing channel was begun, August 6th, 24 400 sec-ft.
Discharge of river on November 4th, when final closing was made, 9 275 sec-ft.
Elevation of water above dam, 113.1 ft. above sea level (C. D. Co. datum).
Elevation of water surface below dam, 97.30 ft. above sea level (C. D. Co. datum).
Total head on closing, 15.8 ft.
Elevation of water surface above dam one week after closing, 112.60 ft. above sea level.
Elevation of water surface below dam one week after closing, 95.85 ft. above sea level.
Total head on dam, November 11th, 16.75 ft.

The tracks were gradually pulled together to a final 13 ft. between center lines, which helped somewhat, but the proper side slopes were chiefly obtained with fire streams, five $1\frac{1}{4}$-in. nozzles, each throwing about 225 gal. per min., being used. The mixed materials as dumped assumed a slope of about $1\frac{1}{4}$ to 1, as a rough average, and

these were very quickly and cheaply flattened down hydraulically to about 2¾ to 1 on the river side and 2 to 1 on the land side. Furthermore, the slopes were really well finished with very slight additional care and expense.

In its final form the dam has about 400 ft. of 15° curve at the north end, and 2 275 ft. of tangent; the dam is connected at each end with the levees extending along the river. At the north end there are 200 ft. of high dam with a rock fill core to within 8 ft. of the top —where it crossed the by-pass. A little more than half way toward the other end there is 600 ft. of another high stretch with a rock core on brush mattress foundation; the remainder is from 16 to 20 ft. high. This is known as the Hind Dam, so called after Mr. T. J. Hind, Superintendent of the work at the Lower Heading after June 1st, 1906, to distinguish it from the Clarke Dam, closing the second break, so called after Mr. C. K. Clarke, Superintendent of the second closing, December 20th, 1906, to February 20th, 1907. About 80% of this dam was complete on December 7th, 1906.

Levee Construction.

The original plans had been to connect the north abutment of the wooden head-gate with the embankment along the river side of the Main Canal, and to build a short section of levee to the south to prevent a flank movement of the river around the diverting dam. The enormous channel which the summer flood of 1906 created in the old Alamo made it obvious that, not only must the break be closed, but that, by a rather elaborate levee system, all overflow water must be kept from getting around into the Alamo. Surveys and examinations showed the necessity of an additional levee from the wooden head-gate to the concrete head-gate, and a levee from the diversion dam south for from 5 to 6 miles. J. C. Allison, Assoc. M. Am. Soc. C. E., Assistant Engineer of the Mexican Co., was assigned to make surveys for these levees on August 1st, and their location was completed early in September. The elevation of the top of the concrete head-gate was 124, and it was decided to put a track over this structure and extend it down the levees, so that the grade was made 126 at Andrade, 124 at the Lower Heading and over the Hind Dam, and thence for 4 miles south, generally 6 ft. above the old high-water marks. At all points the grade was kept approximately 2½ ft. higher than that

PLATE I.
TRANS. AM. SOC. C.V. ENGRS.
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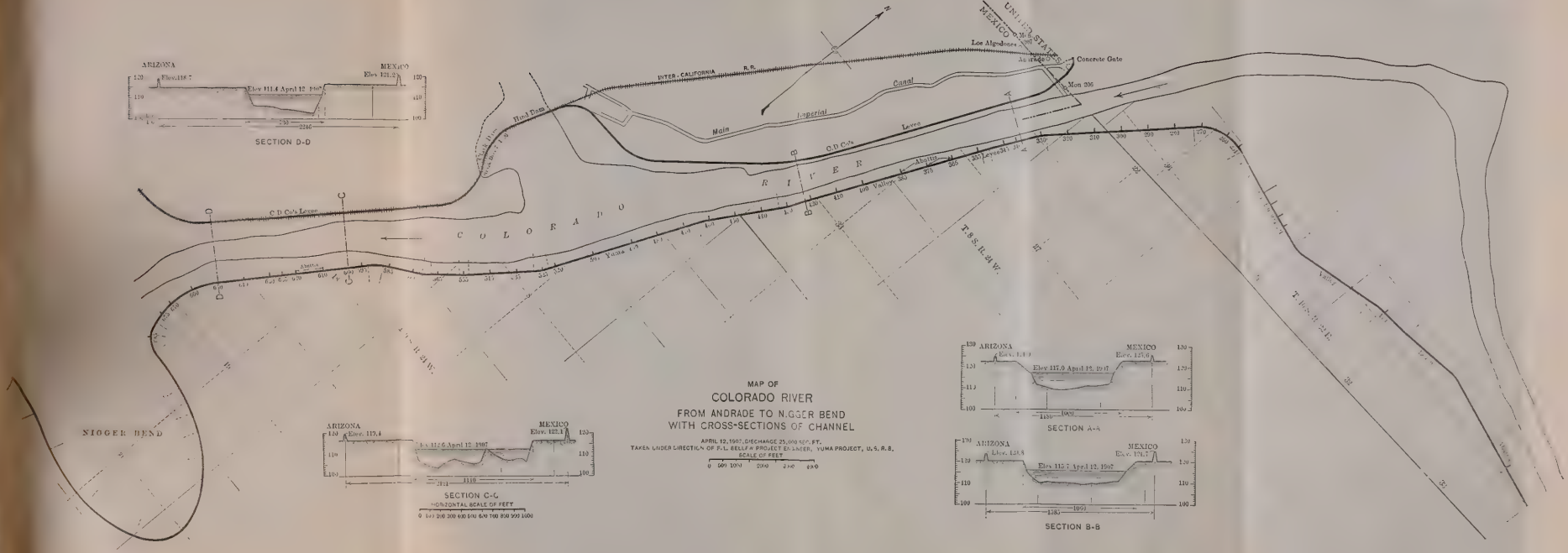
CORY ON
IRRIGATION AND RIVER CONTROL
COLORADO RIVER DELTA.

135-C A.



MAP OF
COLORADO RIVER
FROM ANDRADE TO NIGGER BEND
WITH CROSS-SECTIONS OF CHANNEL

APRIL 12, 1907, DISCHARGE 25,000 SEC. FT.
TAKEN UNDER DIRECTION OF F.L. BELLAW PROJECT ENGINEER, YUMA PROJECT, U.S.R.S.
SCALE OF FEET

of the levee opposite the Yuma Project, because, should the latter break, the damage would be far less than if the levees on the west side were to fail, with re-diversion of the river to the Salton Sea. Between the head-gate and the Lower Heading it was necessary to locate the levee very close to the river, because it must obviously be between the river and the Main Canal, and some large areas of bad adobe, damp, and impossible to work with teams, lay close to the canal and extended well toward the river. Below the break, the levee was also close to the river, because of the similar soil conditions for about 4 miles.

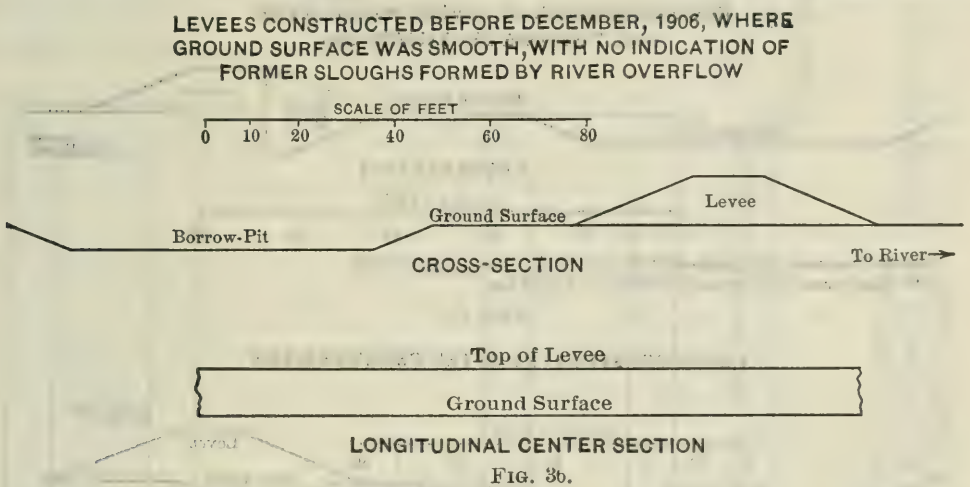

LEVEES CONSTRUCTED BEFORE DECEMBER, 1906, WHERE GROUND SURFACE WAS SMOOTH, WITH NO INDICATION OF FORMER SLOUGHS FORMED BY RIVER OVERFLOW
SCALE OF FEET
0 10 20 40 60 80
Levee
Ground Surface
Borrow-Pit
CROSS-SECTION
To River
Top of Levee
Ground Surface
LONGITUDINAL CENTER SECTION

FIG. 36.

The levee was designed with a top width of 8 ft. and slopes of 2½ to 1 on the river side and 2 to 1 on the land side. The ground for the base of the levee was cleared and grubbed, but no "muck-ditching" was done. The desirability of muck-ditching was fully realized, and it was a part of the levee design. Experience in the valley had always shown that, not only ditch and canal banks, but low borders of irrigated fields, etc., leaked badly when water was first applied. Indeed, interesting cases were cited of water in considerable volume disappearing into the ground for several days, doubtless flowing away under the surface through partly opened cracks of buried layers of cracked adobe.

On the other hand, the money supplied by the Southern Pacific Company was for closing the break, and only for that purpose, until the re-diversion of the river was assured. No narrow construction was placed on this, to prevent building levees at all, but it was not considered proper to incur any avoidable expense in this direction until it should have been clearly demonstrated that it was physically

possible to close the break. No muck-ditching had been done in levee building on the Yuma Project up to that time, and, besides, experience in the valley had always been that cracked adobe layers when thoroughly saturated and under the weight of a few feet of earth soon soften and the underground interstices automatically close. It was thought that the levees could probably be maintained until their bases would thus soak up tight, although it was certain that they would leak like sieves when water first came against them. For these reasons the muck-ditch work in the levee plans was ordered to be omitted.

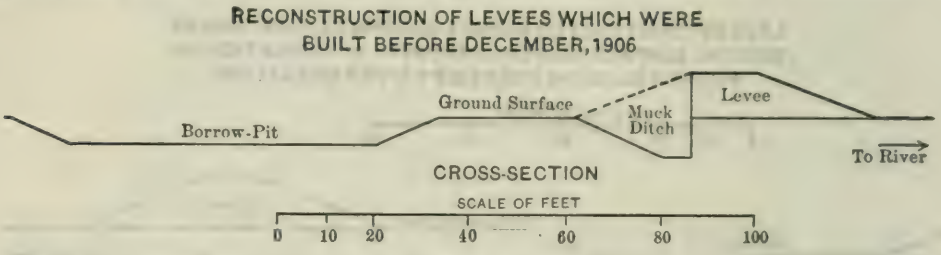

RECONSTRUCTION OF LEVEES WHICH WERE BUILT BEFORE DECEMBER, 1906
Ground Surface
Levee
Muck Ditch
Borrow-Pit
To River
CROSS-SECTION
SCALE OF FEET
0 10 20 40 60 80 100
Levees excavated with muck-ditch to section shown, and then refilled with good material to original section per dotted line

FIG. 37.

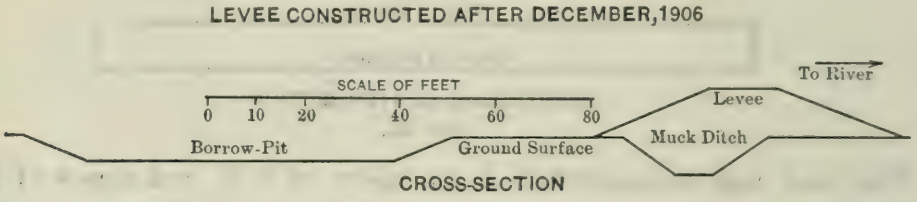


FIG. 38.

Material for the levees was taken from borrow-pits on the land side. It was fully realized that this was not in accordance with the usual practice, but it was decided on after careful consideration of the advantages and disadvantages. The location of the levee was forced very close to the river for a great portion of the way, and the levees of the Yuma Project on the opposite side were also so close to the stream that the distance between was in many places only 1 400 ft.—an exceptionally narrow waterway for such an unruly stream as the Colorado. As it was certain that the current at flood stages would be very great in such sections, it was extremely desirable not to disturb in any way the rank vegetation between the river and the levee, as it could not help but greatly break up and retard currents and thus protect the levee from erosion.

Experience with the levees of the Yuma Project showed that the hope that borrow-pits would be silted up was vain, and instead, that

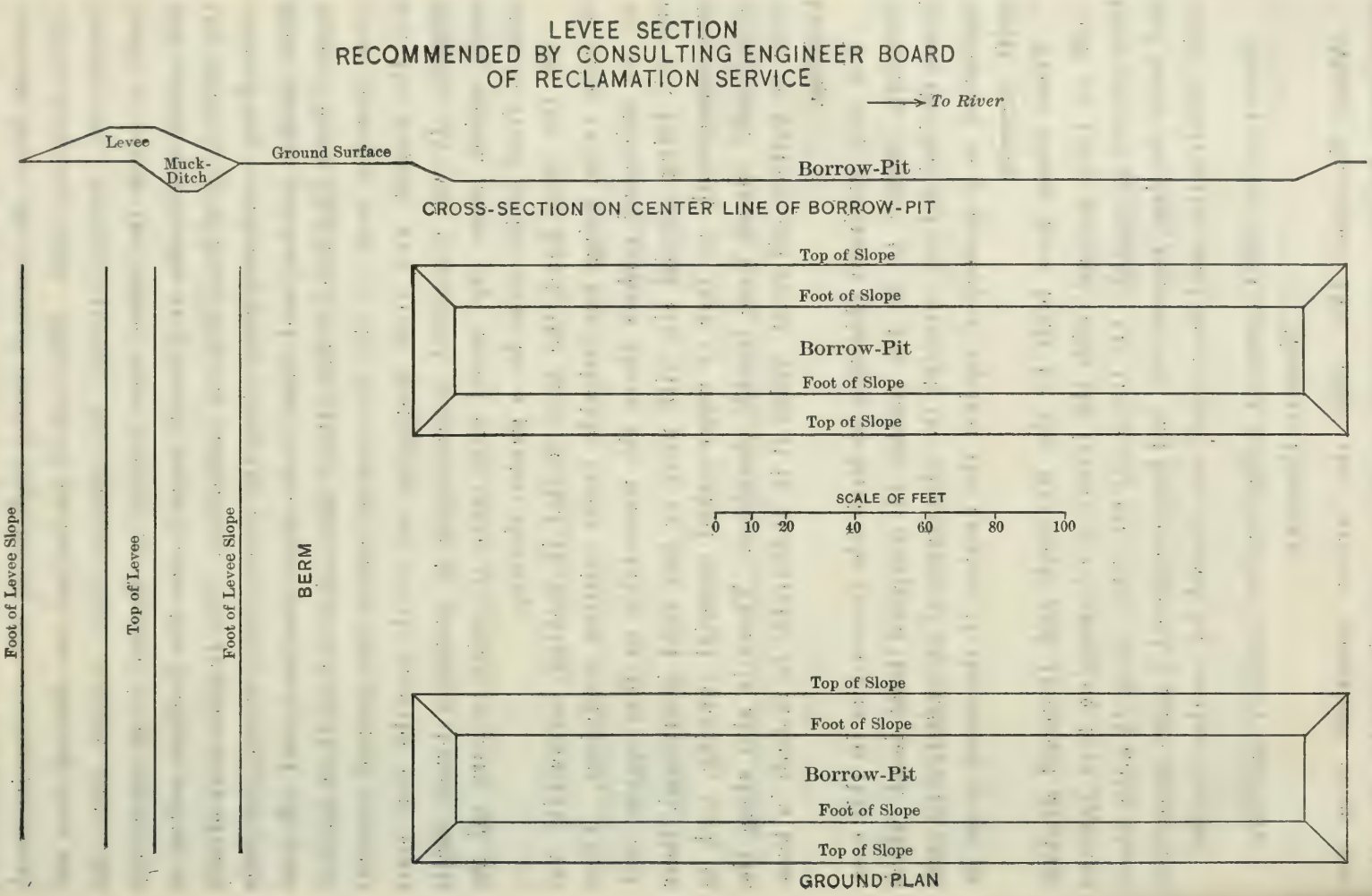

LEVEE SECTION
RECOMMENDED BY CONSULTING ENGINEER BOARD
OF RECLAMATION SERVICE
To River
Levee
Muck-Ditch
Ground Surface
Borrow-Pit
CROSS-SECTION ON CENTER LINE OF BORROW-PIT
Top of Slope
Foot of Slope
Borrow-Pit
Foot of Slope
Top of Slope
SCALE OF FEET
0 10 20 40 60 80 100
Foot of Levee Slope
Top of Levee
Foot of Levee Slope
BERM
Top of Slope
Foot of Slope
Borrow-Pit
Foot of Slope
Top of Slope
GROUND PLAN

FIG. 39.

they would be cut together to form a continuous canal having eddying currents below the traverses during high floods, unless extensive brush abatis work was used. This sort of protection was deemed very unsatisfactory, because, though the Mexican Co. actually owns the land on which the levees were located in Mexico, it is practically impossible to exercise very much control over the Indians, owing to the indifference of local Mexican authorities. The Indians have always utilized any overflowed areas along the river as they wished, for their little garden patches, and these levees must absolutely cut off such water. For a long time it was utterly impossible to keep these nomads from planting seed in the borrow-pits, where the ground remains wet the longest, and from destroying all brush growths that start therein. It was considered impracticable to maintain brush abatis work, which, when dry, would only make it easier to burn off the area in front of the levee for a garden clearing.

On the other hand, the land spoiled in making borrow-pits was of little value, being non-irrigable under existing conditions. There are no quicksand pockets above the water-table in that region, and, the soil being alluvial silt with more or less sand intermixed, there was consequently no fear of water-soaked material running, such as occasionally causes levee trouble elsewhere. There is also along the river no surface soil crust, which it is undesirable to disturb in levee building.

The only pertinent objection to land-side borrow-pits in this case, therefore, seemed to be the matter of increased total head, which it was decided did not outweigh the advantages of an undisturbed rank vegetation as a protection against the erosion of the water slope by swift currents.

These levees were built by the Shattuck and Desmond grading outfit on force account, with the intention of changing to a yardage basis as soon as possible. On December 6th, 1906, about 1½ miles above and below the dam, respectively, had been completed, 5 miles more were under construction, and the ground was cleared for another 2 miles.

Situation of the California Development Company and the Mexican Company.

About November 15th, 1906, the various operations along the river were making satisfactory progress, and the writer for the

first time since June 1st left the river, hurriedly investigated the condition of the C. D. Co. and the Mexican Co. and reported his findings toward the end of that month. As a result of this, when the second break occurred a few days later, he advised against further advances by the Southern Pacific Company, and President Randolph concurred and so reported to Mr. Harriman. The fact that such a decision was made by the Harriman interests unqualifiedly has not been generally accepted without some mental reservations. It may be interesting, therefore, to make some excerpts from this report. The balance sheet for the combined C. D. Co. and Mexican Co. was approximately as follows:

ASSETS.

Real estate (chiefly in Mexico)		$545 037.26
Stocks (chiefly unsold water stock)		175 600.00
Plant:		
Machinery and equipment	$179 621.82	
Branch railroad track	63 000.00	
Canals in Mexico	375 000.00	
Canals in United States	308 616.37	
		926 238.19
Accounts receivable (chiefly notes secured by water stock)		235 137.02
Total		$1 882 012.47

LIABILITIES.

S. P. Co.—Audited bills and interest		$1 532 595.73
General audited bills and interest		73 786.72
Bonds and accrued interest		515 200.00
		$2 121 582.45
Damage claims (probable):		
New Liverpool Salt Co	$50 000.00	
Land owners	200 000.00	
Water Companies Nos. 6 and 8	500 000.00	
S. P. Co	1 000 000.00	
Inter-California R. R.—S. P	250 000.00	
		2 000 000.00
Total		$4 121 582.45
Net liabilities		2 239 569.98

The possibility of extending the canal system and selling additional water rights was discussed, and the opinion was offered that such possible returns would probably just about suffice for building a canal from the river into the valley, to take the place of the Alamo channel, which might be necessary in 5 or perhaps not for 20 years; and for building new controlling works on it in the valley.

The conclusions were that maintenance and operation expenses, properly estimated, would take up the returns from the sale of 600 000 acre-ft. per annum. The cost of protecting the region from the Colorado flood-waters was set down as problematic, probably averaging $100 000 per year, and fluctuating enormously, and it was advised that the task of controlling the river was too serious a tax on the enterprise under any possible circumstances, while international complications would probably mean considerable delay in arranging for such work equitably and satisfactorily.

The Second Break.

On December 5th, 1906, a severe flood came down from the Gila, as shown by Plate XLVII. Superintendent Hind and the writer, who were in Imperial Valley at the time, received telegraphic notice from the water-shed, and went at once to the river. For reasons already explained, trouble was expected from water getting under the levees, because no muck-ditch protection had been provided, and a large force of men was detailed to watch the sections for a mile on either side of the Hind Dam day and night as soon as water came even near the river toe of the levee. Information from the upper stations throughout the Gila water-shed frequently indicate floods which never materialize, and this was another case of the "truth itself" being not believed. There was so much accumulated work in the valley that there was no time to watch a discharge of 30 000 sec-ft., although a river stage which would test out and soak up the levees was obviously of great importance. Mr. Hind and the writer were in Yuma, returning to the valley, when the flood reached the Lower Heading, where the river began rising at midnight and rose at the rate of 1 ft. per hour until the peak was reached early in the morning. At 3:30 A. M. Mr. Hind and the writer left Yuma on the work train, reaching the Lower Heading about 5:15 A. M., and found three serious and distinct breaks within 100 yd., the first one being about 2 400 ft. from the south end of Hind Dam. In addition water was finding its way under the levee in

about ninety other places, within the stretch where the water reached the toe of the levee, or about ¾ mile above and an equal distance below the dam. Mr. J. Calvert, General Foreman, had fully obeyed instructions, and when the water began to reach the toe of the levees at the lowest point, had commenced work with his force of about 75 men, doing all that seemed possible. The trouble was not that any one break could not have been easily handled, but that so many points of weakness were developed practically at the same time. Indeed, it is really remarkable that the situation got beyond control in only these three places.

A part of the general arrangements with the Inter-California Railroad was its use of the Hind Dam. On the south end of this dam the proposed alignment turned a small angle to the right, and it was planned to have the fill for the next 2 miles without any openings—constituting thus a spur levee to prevent any water which passed through the main levees from reaching the old channel of the break beyond the dam. Had this fill been ready, or had the first serious break been 500 ft. farther along, sand bag diking across the traverses and out into the brush to force the water to spread out and follow an old and well-defined swale which entered the Alamo 2 miles beyond, would have been easily possible. In either case, the damage would have been limited to losing less than 1 000 cu. yd. of levee section.

As it was, however, the water coming through the breaks filled the borrow-pits on the land side, overflowed the intervening traverses, and, as the land in general sloped westward, over-topped the last traverse by the channel of the break below the dam, and caused a rapid grade recession from there, following the borrow-pits through the nearest break. When Mr. Hind and the writer reached the scene, the first of the three breaks was beyond control, and the situation was hopeless.

By the time the grade recession had reached and passed through the break, the flood had crested, and the water had risen against the levee to a depth of about 4 ft. The water rushing through rapidly increased, and cut into the far side of the bank, and was deflected and began cutting into the land side of the levee. This soon breached the dike about 1 000 ft. from the end of the Hind Dam. This breach became the main break, and was rapidly widened and deepened until, within 24 hours, the old channel was again entirely dry and the river had been re-diverted into the Salton Sea.

The men of the grading outfit engaged on levee extension work 3 miles down the river were flooded out, and the steamer *Searchlight* was sent to relieve them. The re-diversion into the Salton Sink occurred so rapidly that the steamer was left grounded there in the old channel, and inasmuch as it was the only craft on the river not controlled by the contractors on the Laguna Weir, this was a serious matter.

The Southern Pacific Quits.

This disaster brought to the higher authorities of the Harriman Lines a thorough realization of the size of the great task of controlling the Colorado. The imperative need of invulnerable levee construction for at least 10 miles along the river was made evident, and the difficulty and cost of building and maintaining such a system was emphasized. Entirely aside from the very great cost of bank protection work to prevent the breaching of levees through the side cutting of the banks, was the difficulty of building in such bad soil a line of defence which would be absolutely dependable.

The financial condition of the C. D. Co. and the Mexican Co., as just explained, was very bad, and, under the most favorable circumstances possible, the chances of the Harriman interests ever being able to get back very much of the moneys already advanced were extremely remote. In addition, however, it was apparent that an unusually efficient and expensive levee system would be required, the first and maintenance cost of which was too large a burden to undertake.

The stockholders of the irrigation properties notified the Southern Pacific management controlling them that the properties could not be expected to do such overflow protection work, and indeed should not pay more than a proportional part based on the total value of the property interests in jeopardy, especially in view of the immense amount of work which had already been done at its expense. Urging that the irrigation company had caused the menace (which may or may not be entirely the case) had not the slightest significance to the Southern Pacific interests, which were really the only ones with any funds, collateral, or equipment, and were in no possible sense responsible for any changes in physical conditions along the Colorado River, except to make them very much better than they otherwise would have been.

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FIG. 40.—LEVEE FAILURES RESULTING IN SECOND BREAK, DECEMBER 7TH, 1906. NOTE THE SHORT LENGTH OF THE TWO GAPS.

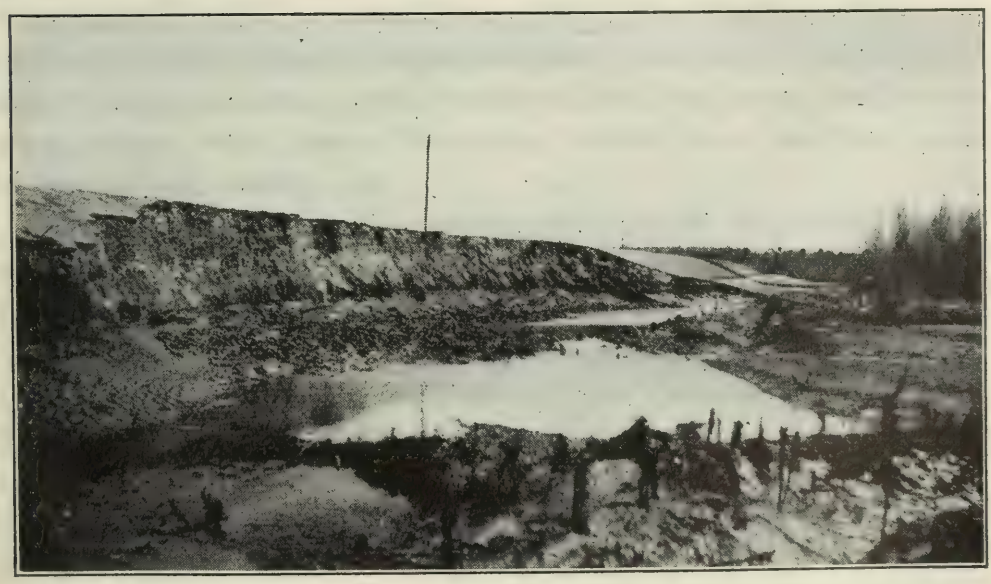


FIG. 41.—EFFECT OF FLOOD OF DECEMBER 7TH, 1906, ON LEVEE OF U. S. RECLAMATION SERVICE, OPPOSITE LOWER HEADING.

On the other hand, it was recognized that something would have to be done very quickly because the summer flood of 1907 would in all probability cause such grade recessions as to force a hurried exodus from Imperial Valley which would be without a parallel in history. The chances of such grade recession extending far enough to render the control of the river after that flood very much more difficult, were remote. The matter was made complex by the fact that all work had to be done in Mexico and practically all property interests in jeopardy were in America; and there were no provisions, State or National, to handle such a curious situation. Unless the river was turned and kept going to the Gulf, the Southern Pacific would suffer the loss of its traffic from the Imperial Valley and would have to change its line and build 100 miles of track, but, to obviate these losses would certainly not justify it in undertaking to control the river single-handed.

Accordingly, the people of Imperial Valley were notified that, while the Southern Pacific would be very glad to place such equipment and organization as it had along the river at the disposal of any party who wanted to proceed with the work, and would be willing to contribute toward the expenses thereof in proportion to the value of its interests as compared to all others in jeopardy, it would not advance additional funds without a definite arrangement for being reimbursed. Work on a roadbed following the —100-ft. contour was ordered and rushed to completion. The cost of grading was very small, and such a line would preclude the possibility of the interruption of transcontinental traffic by the Salton Sea for at least 4 or 5 years, during which time a line lying entirely above sea level could be economically constructed.

On December 13th a mass meeting of the people of Imperial Valley was held in Imperial, and subscriptions for river control work, totaling $950 000 were made by various interests. These were the Imperial Valley Improvement Company (the practical successor of the Imperial Land Company) $100 000; the Holton Power Company, $100 000; the C. M. Co., $250 000, and the directors of the mutual water companies together a bond issue of $500 000. All these were made promising payment 90 days after the break should have been closed successfully, the railroad to assume all risk of the work.

While considering these subscriptions, it was urged in opposition that the mutual water companies might not be able legally to issue

bonds or expend money for river protection work at all, or indeed that the people of the valley could raise money, except by individual subscription, for work to be done in Mexico. Requests were sent out in all directions, resulting in numerous civic and political bodies and authorities of the State wiring to President Roosevelt asking to have the United States Government act in the emergency. The President acted promptly, and as the result of telegraphic correspondence with Mr. Harriman, instructions to start work on the river were received on December 20th.

In the meantime the organization at Andrade and at the Lower Heading had been kept intact. The quarry was developed, sidings just across the border in Mexico were lengthened to 7 000 ft., and material and equipment of all possible kinds which might be needed were gathered in readiness to proceed whenever orders might be received.

Senator Flint's Bill.

Immediately after the holidays, Senator Frank C. Flint, of California, introduced a bill in the Senate providing for the appropriation of $2 000 000 to handle the situation. Under the provisions of this bill, whatever sum might be found just should be paid to the Southern Pacific Company for work then under way, and the remainder should be utilized to establish an irrigation project for Imperial Valley by the U. S. Reclamation Service. The idea was that the irrigation of American land in the Salton Basin and the regulation of the Colorado River were inseparably connected, and that as soon as the situation should be under control by the Southern Pacific Company, the entire matter should be turned over to the Reclamation Service for future handling. Director Walcott sent Professor A. E. Chandler, of the University of California, then with the Reclamation Service, to the scene to make a special report, and President Roosevelt, on January 12th, 1907, sent a special message to Congress severely criticizing the promoters of the C. D. Co. and the management of the properties, and urging the passage of the Flint bill in order to relieve the settlers of Imperial Valley from the "injustice" they were enduring.

When the bill reached the House, Hon. S. C. Smith, Representative from the Eighth California Congressional District—in which Imperial Valley is located—opposed it, advising that he did so because of re-

quests from his constituents in the valley. There can be no doubt that, with very few exceptions, the farmers in the valley objected to the bill, preferring the existing irrigation arrangements to those which would follow under the Reclamation Service,* and desiring governmental assistance in river protection work, and in that only. Largely due to Mr. Smith's efforts, the bill failed to pass.

Change in Organization.

In spite of the opinions of visiting engineers, experience during the first closing left little doubt that there would be any particular difficulty in making the second closing without any brush mattress foundation. Even had such been deemed desirable and worth the delay in time, very little brush was available nearby because of the large quantity used for the first closing. It was felt that the work was standardized and consisted in throwing ordinary railroad trestles across the break and from them making rock fill dams in series. The levee construction work to be done, however, was very much of a problem in every way. Superintendent Hind was transferred to the levee reconstruction and extension, and Mr. Clarke, formerly Resident Engineer of the Tucson Division, Southern Pacific Company, came to the work as Superintendent of the second closing on December 20th, 1906.

At the same time, an entire change in the accounting system was ordered, effective December 7th, the date of the break. Prior to that time the work had been done by the Mexican Co., with material and funds supplied by the C. D. Co., and the latter corporation from time to time borrowed money from the Southern Pacific Company. This was changed so that the Mexican Co. was furnished money by Epes Randolph, Agent of the Southern Pacific Company, and the C. D. Co. had nothing to do with the matter whatsoever. On the American side of the line, the operations were exclusively under the name of "Epes Randolph, Agent, S. P. Co." The railroad furnished supplies and material to him under the same arrangements and conditions as it had furnished them to the C. D. Co., namely, at cost plus 10 per cent. One marked difference, however, occurred in making these charges, namely, that after January 1st all freight bills were rendered at traffic

* Among the objections, the two most important were the probable increase in cost of water and the necessity for reducing individual holdings probably to 40 acres or less.

rates instead of at ½ cent per ton-mile, this being made necessary by the provision of the Interstate Commerce Commission which took effect at that time.

Closing the Second Break.

The work of closing the second break was in several ways interesting. To begin with, the current struck where the south end of the Hind Dam had been, and was there deflected sharply, resulting in very serious erosion. Few people who saw the break at this stage believed it possible to hold this erosion from going entirely through the structure, but by unloading immense quantities of rip-rap, the fill was held, the water in front cutting to a depth of about 42 ft.

Two trestles were decided on and started, five pile-drivers being used, one at each end of each trestle and a floating machine in the middle of the stream. These trestles had to be thrown in a curve concave up stream to connect with the levee on the south side, therefore the piles had to be driven in a very strong quartering current. The channel was narrowed and the bottom cut down to a maximum depth of 38 ft. In driving 90-ft. piles under these conditions, there was constant danger of overturning the driver and losing the machinery, therefore two complete pile-driving outfits were kept in reserve and two boats waited below the trestle to pick up any men who might be thrown overboard.

On December 28th one line of trestle was practically completed, when a flood, shown on Plate XLVII (which, by the way, let the stranded steamer *Searchlight* get back up the river to the work), carrying unusual drift, tore out about one-third of it. Three times this occurred, resulting in the loss of a large quantity of bridge material. All this was obtained through Mr. R. H. Ingram, General Superintendent, Southern District, Southern Pacific Company, and the following telegram from him is interesting:

"Los Angeles, 1/14/07.

"H. T. C.—

"We have exhausted all available supply of piles in San Diego and Southern California. There is very little hope of getting any in Northern California. If you feel that you will need any more please let me know at once as we must make arrangements with the Atlantic System.*

"R. H. Ingram."

*The lines from New Orleans to El Paso.

On January 26th, 1907, the first trestle was finished for the fourth time, all stringers were in place, and the track was two-thirds laid. In the second trestle, 50 ft. above the first, seventeen bents remained to be driven. The fill on the south, connecting the trestle with the levee, was 60% completed. Stored on the branch line in Mexico and the United States there were 175 "battleships" of rock, loaded at the quarry of the C. D. Co. by steam shovels, and 100 flatcars of large rock from the distant quarries. At each end of the dam, ½ mile of the levees had been reconstructed, and 1½ miles more opened up, while there were 1 100 men and 1 000 head of stock engaged at Andrade and on the river.

On January 27th, the first trestle was completed, and dumping rock from it began at 5 P. M. By daylight 145 "battleships," containing 6 600 cu. yd. of rock had been unloaded as another flood from the Gila began arriving, but it caused little trouble. On February 10th at 11 P. M. the second break was closed, and all the water was again going down the old channel.

The following materials were used in the second closing:

4 000 ft. of railroad trestle—of this, 1 800 ft. were carried away by floods before either trestle could be completed or any rock could be dumped. When rock dumping began, no more trestle was lost; the final result was two trestles, 50 ft. apart, and each 1 100 ft. long, or 2 200 ft. of trestle.

16 000 ft. of 8 by 17-in. Oregon pine stringers—8 000 ft. of these were removed.

1 200 piles.

45 000 cu. yd. of earth placed by teams—making 960 ft. of earth dam to connect with the levee, 31 000 cu. yd. being placed by February 2d.

2 157 car loads, or 55 000 cu. yd., of rock used prior to actually closing off the water.

221 car loads, or 7 735 cu. yd., of gravel.

203 " " " 8 840 " " " clay.

The discharge of the river when work began on December 20th, 1906, was 12 500 sec-ft.

Dec 31st	48 900	sec-ft.
Jan. 7th	15 200	"
Jan. 12th	44 300	"
Jan. 18th	16 300	"
Jan. 20th	33 400	"
Jan. 27th (when first rock dumping began)	13 800	"
Feb. 3d	31 300*	"
Feb. 7th	17 700	"
Feb. 11th	20 800	"

After the break was closed, 956 car loads, or 38 240 cu. yd., of clay and 873 car loads, or 33 555 cu. yd., of gravel were used to complete the Hind-Clarke Dam. The rate of dumping rock, etc., is shown by Table 14.

TABLE 14.—CAR LOADS OF MATERIAL.

	"Battleship."	Flatcar.	Clay.	Gravel.
January 27th to February 2d	606	501	8	15
February 3d to 9th	489	385	58	140
February 10th to 10th midnight	85	91	38	28
	1 180	977	104	173
February 11th to 15th	323	77	99	48
	1 503	1 054	203	221

The water was shut off on February 10th at 11 P. M. and the flatcar rock then on hand and *en route* was unloaded in order to release the cars, the rock from Andrade being used until the 18th in raising the dam hurriedly to protect it against possible floods. The total car loads of material used in the closing and in the finishing up of the Clarke Dam were:

"Battleship."	Flatcar.	Clay.	Gravel.
1 490	1 182	784	873.

During the entire operations subsequent to the second break, including finishing up the Hind-Clarke Dam complete, but exclusive of the rip-rap to hold the grade before the completion of the trestle over the break on January 27th, the following car loads of material were used:

"Battleship."	Flatcar.	Clay.	Gravel.
1 517	1 240	956	2 052

*4 300 sec-ft. down the old channel.

The clay pit was closed on March 3d, and the Hind-Clarke Dam was finished, except for the final surfacing with the spreaders, on March 15th.

Two steam shovels, and part of the time three (not including the one in the Mammoth gravel pit) were engaged from September, 1906, to May, 1907, at $7.50 per day, prior to January 1st, 1907, and at $12.50 thereafter. During the period, there were from 1 to 12 locomotives, at the following rates:

American, light	$ 8.00 per day.
Moguls	10.00 "
Consolidation	12.00 "
Car pile-drivers, working 40 days at	10.00 "
Donkey engines	1.50 "
Skid-drivers, complete	2.50 "

TABLE 15.—Southern Pacific Equipment used on River, from December, 1906, to July, 1907, Expressed in car-days.

Month.	Ballast cars, at 25 cents per day.	Steel side dump cars at 50 cents per day.	Cabooses, at 50 cents per day.	Water cars, at 50 cents per day.	Outfit cars, at 30 cents per day.	Home freight car detention (box and flat), at 15 cents per day.	Foreign freight equipment detention.
December, 1906	595	3 271	80	141	42	371	306
January, 1907	690	4 972	30	174	....	675	600
February, 1907	219	4 609	15	153	....	47	78
March, 1907	54	5 565	27	321	....	47	71
April, 1907	....	5 255	4	323	....	28	2
May, 1907	2	4 792	9	440	....	17	2
June, 1907	....	2 635	53	264	....	32	....
July, 1907	....	1 336	12	24	....	....	....
Totals	1 560	32 435	230	1 840	42	1 217	1 059

The Clarke Dam extends from the south end of the Hind Dam across the second break to the levees on the south. These two dams, consequently, constitute one continuous structure, which is known as the Hind-Clarke Dam.

Results of Experience in Constructing the Hind-Clarke Dam.

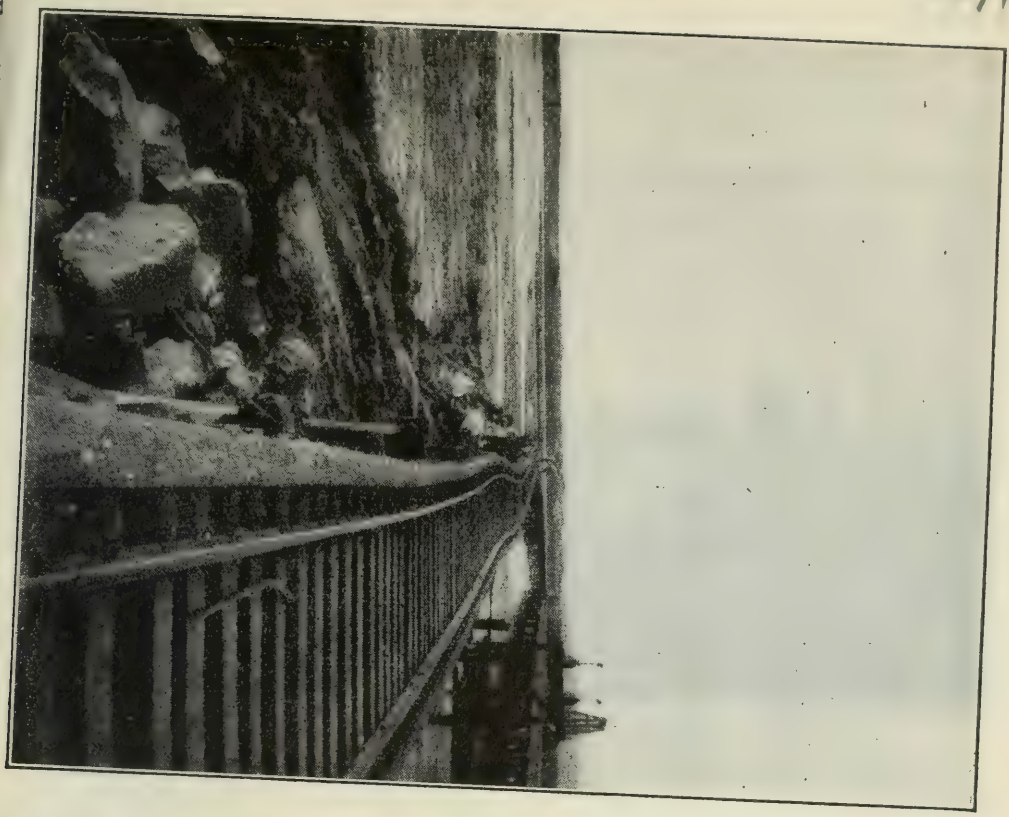
The experience obtained in making these two closings, according to the methods used, afforded some information regarding work of this class which is believed to be entirely unique and in some respects

unexpected. In the first place, it was shown that the brush mattress bottom protection is not only unnecessary, but adds to the cost, both in time and money, provided rock is thrown in at a reasonably rapid rate. In discussing the possibility of handling the situation along the lines decided on, the opinion was freely expressed that the rapid current would carry smaller rocks indefinite distances down stream, and that the larger ones would quickly settle into the soft, water-soaked, alluvial soil to indeterminate but very great depths. As a matter of fact, a relatively small quantity of "battleship" rock sufficed to blanket the bottom of the stream with a mattress of rock which fulfilled essentially the same function as a mattress of brush.

In this type of construction it is desirable to have rock of various sizes, such as obtained by blasting large quantities in the quarry and loading with a steam shovel. Large stones (from 1 to 7 or 8 tons), which can be loaded on flatcars only with derricks, are effective, but not absolutely necessary in raising short sections where, through carelessness or a little unexpected settling, an unusual quantity of water is going over a low place in the dam with consequent menacing velocity. Such large rocks were unloaded by a great number of men using pinch-bars, and, to prevent upsetting, the cars were chained to the stringers when unloading especially heavy rocks. One man was killed during each closing by going overboard with a large rock, and these were the only serious accidents. No equipment was lost during either closing.

Two trestles were used in both cases. These were 30 ft. apart in the Hind Dam, and 50 ft. apart in the Clarke Dam, the idea being that the current would prevent building a rock mattress extending far enough up stream by merely dumping rock from the trestle on which the closing was made. Careful examination of the resulting cross-section, when the water was shut off the first time, seemed to indicate pretty clearly that it was a needless precaution to have two trestles for final heads of 14 ft. or less. As there was to be no mattress in the second closing, it was decided to use two trestles, in order to take no avoidable chances. Everything worked so well that it seemed safe to do a little experimenting, and, practically speaking, the second break was closed from the lower trestle alone.

In both cases the fills at both ends were kept well above the possibility of being overtopped, and of uniform heights across the re-



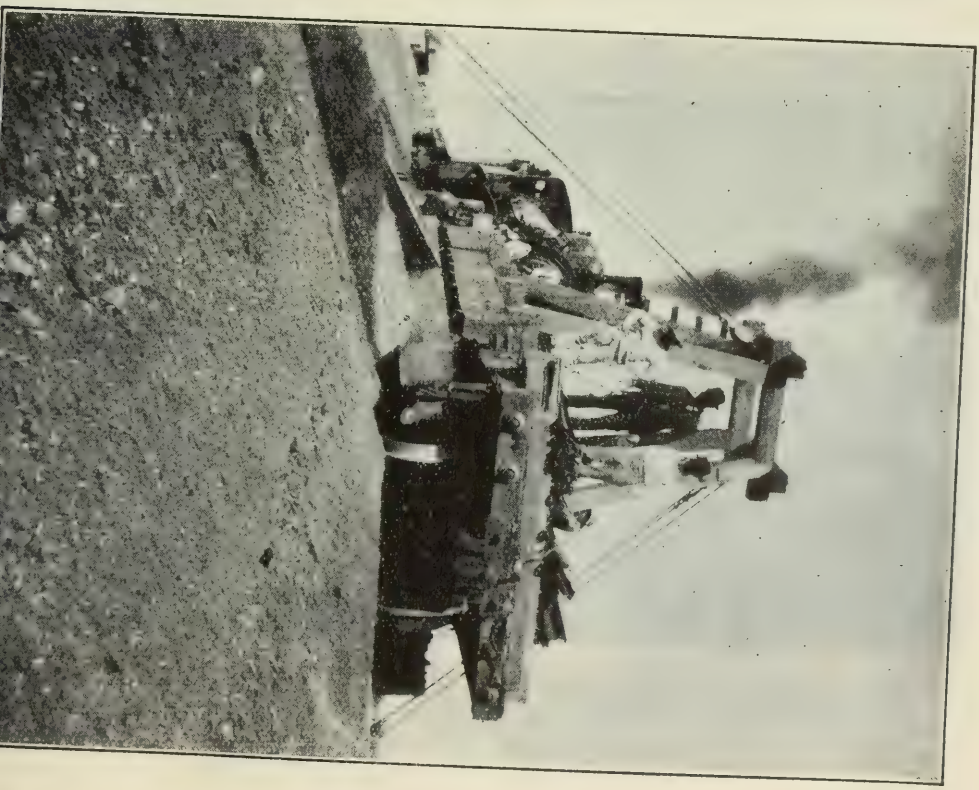


FIG. 46.—FAILURE OF ROCKWOOD GATE, OCTOBER 11TH, 1906.

FIG. 47.—MUCK-DITCH CONSTRUCTION IN LEVEE EXTENSION WORK, 1907.

maining length. Train loads extending entirely over the trestles were unloaded most of the time; but short sections of the fill which were low were promptly filled in, and great care was exercised to distribute the overpour evenly. Once, in the building of each of these structures, a local settlement occurred, resulting of course, in large quantities of water going over the relatively short lowered section. The same experience was had in making the upper coffer-dam wall for completing the gap in the Laguna Weir. The construction of a rock fill barrier dam was regarded as impracticable by engineers, because of just such occurrences surely breaching the barrier hopelessly. In point of fact, continued dumping of even small rock soon stops such so-called breaks, although that results in the waste of much material.

The chief difficulty is caused by drift in the stream being caught by the trestle. Theoretically, such drift should be very easily broken up and carried to the bottom by dumping rock. In practice, however, there is difficulty in keeping drift accumulations from seriously threatening the trestle. In Fig. 44 a very sharp kink will be observed which is due to drift accumulations throwing the entire trestle out of line. Two or three bents were taken out of the upper trestle from this cause.

It was found that a cross-section of the dam under the trestle is about as shown in Fig. 48, so that it is the inevitable little breaks and slides, rather than actual settling, which occurs in such cases. Local readjustments or settlings were easily handled with an estimated quantity of 100 sec-ft. per lin. ft. of dam going over the rock fill. How much more could be handled safely with the rock available is not known. The writer would say that experience along the Colorado River would not justify using the rock fill barrier dam method for quantities in excess of half that flow, on an average, although possibly much greater rates of discharge over such a structure might be safe. The quantity which went over the 680-ft. crest was 27 000 sec-ft.

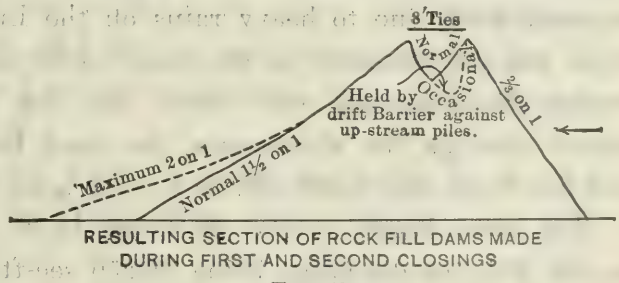


Fig. 48.

The total length of rock fill dam used in the trestle was 1 125 ft., and the water was allowed to pour over 680 ft. of it; the trestle consisted of 4-pile bents, all piling vertical, 15-ft. spacing

between bents, average penetration of piling about 18 ft. The total head, or the difference in water surface above and below the dam, was 12.45 ft., all developed in place in 13 days and 5 hours.

By way of comparison, the building of the upper and lower sides of the coffer-dam within which the central or channel portion of the Laguna Weir was completed is most interesting. One line of trestles, 740 ft. long, was thrown across the stream just above the upper edge of the weir, and another trestle, 728 ft. long, below the lower edge, the trestles being about 400 ft. apart. From these trestles rock fills were built up, and all the water in the river was forced through the sluice-gates at the ends of the weir, the Arizona sluice-way being 2 700 ft. distant and the California sluice-way 1 500 ft. There were well-developed quarries at each end of the weir, and the haul was from 1 500 to 3 500 ft. The following equipment was used:

One 2½-cu. yd. Atlantic type steam shovel (in California quarries).
One 2½-cu. yd. Marion steam shovel (also in California quarries).
Seven 12½-in. American steam hoists, mostly in the Arizona quarries.
Six 18-ton standard gauge Davenport locomotives.
Fifty 6-cu. yd. dump cars.
Eighteen 5-cu. yd. steel dump cars.
Fifty flatcars.

By December 10th, openings to and from the sluice-ways were completed, and the rock fills were advanced until the head was 2.8 ft. On December 19th, the head had been increased to 6.9 ft., of which about 3.5 ft. were on the lower rock fill, while 8 500 sec-ft. were going over these and 3 200 sec-ft. through the sluice-ways. On account of a reported flood, due to heavy rains on the Little Colorado water-shed, work was concentrated on the upper coffer-dam, and by noon of December 21st the gap was closed and the river—9 950 sec-ft.—was turned through the sluice-ways, the head being 3.5 ft. on the lower and 5.4 ft. on the upper fill, or a total head of 8.9 ft.

The river began to rise 30 hours later, and crested at daybreak on the 23d, the discharge being 35 250 sec-ft. Water passed over the entire length—4 200 ft.—of the completed dam, 3.2 ft. deep, but lacked 10 in. of reaching the general level of the upper rock fill. The lower coffer-dam was injured considerably by water running length-

vise of the dam on the down-stream side, so that the upper rock fill tood all the head, which reached a maximum of 10.1 ft. Two weeks ater the damage sustained by the railroad tracks on top of the finished arts of the structure had been repaired, the lower coffer-dam had been completed above the danger line, and work had commenced on he 650-ft. gap in the main structure within the coffer-dam.

About 600 men were engaged on the closing, working in two 10-our shifts. The total cost of turning the river through the end sluice-gates and constructing the coffer-dam is given as $86 072; the total ock used was 59 750 cu. yd. of excavation, making 82 800 cu. yd. n the rock fill. No lives and no equipment were lost, and the time equired after beginning to dump rock was about 2 weeks.

The total cost of the rock work in the fills, including trestles, quarrying, train service, superintendence, depreciation, and all over-ead charges of the project, was $1.04 per cu. yd., of which 45%, or 38 720, was for excavation, Class 1 (quarrying solid rock).

Completing the Clarke Dam.

The Clarke Dam was rendered impervious by dumping Mammoth it gravel and clay from both trestles. No attempt was made to puddle r settle the material by hydraulicking, as was done with the Hind Dam. Small local settlements occurred from time to time for a year fter the dam was completed, but nothing disquieting in any way. mperviousness was very quickly obtained, indeed, long before the tructure was raised to grade and widened to its proper dimensions.

In constructing the Hind Dam, every effort was made to hurry he work, and the stringers were not taken out. With the Clarke Dam, ll stringers were removed, the tracks were raised and narrowed to 3 ft. from center to center, and the top and sides were finished off vith the gravel spreader used on the levee work, leaving the finished tructure as shown by Fig. 50.

Spur Levee.—A spur levee, 8 700 ft. long, was built, starting at he elevation of the top of the Hind-Clarke Dam (124 ft.), and with n initial descending grade of 0.5 per cent. The purpose of this evee is to prevent water from any break in the main levee south of he Hind-Clarke Dam from getting into the old dry channel below, s happened when the second break occurred. It is intended to hold uch water back and make it spread over the low country in a sheet.

This levee, which was decided on, but not started, before the second break, is located along an old overflow channel slightly higher than the country on either side, and where test-pits showed need for little muck-ditching. Arrangements were made later with the Inter-California Railroad to change its alignment to use this spur levee and the Hind-Clarke Dam, and it was thus extended by this railroad grade for 4 miles without any opening.

Changes in Staff.

The success of the Hind-Clarke Dam being assured, arrangements were concluded whereby F. C. Herrmann, M. Am. Soc. C. E., on February 1st, 1907, became Assistant Chief Engineer of both the C. D. and Mexican Cos., particularly for the purpose of making surveys and estimates for reconstructing and extending the canal system in Imperial Valley. About March 1st, the Clarke Dam being then well advanced, Mr. Clarke was transferred to the valley and appointed Superintendent of Construction of both companies, Superintendent Hind remaining in charge of all operations along the river until they were finished, when he was transferred to the Harriman Lines in Mexico, on August 1st, 1907. On May 1st, Mr. Clarke returned to the railroad as Resident Engineer of the Coast Division. Nearly two years later he came back to the valley as Superintendent of Imperial Water Company No. 1, and early in January, 1910, was appointed by the Receiver of the C. D. Co. as Assistant General Manager and Chief Engineer, which position he resigned in April, 1911. The writer remained as General Manager of the properties, but relinquished the title of Chief Engineer in both companies in July, 1908, and issued circulars advancing Mr. Herrmann to the positions which he held until he left the companies, in March, 1910, after the appointment of a receiver. His successor in the Mexican Co., and its present Chief Engineer, is Mr. C. N. Perry, who first came to the valley with Mr. Rockwood in 1892, and was Resident Engineer of both companies from October, 1901, to August, 1906, and so was in immediate charge of most of the existing canal construction in Imperial Valley. Since April, 1911, Mr. J. C. Allison has been Chief Engineer of the C. D. Co.

Levee Reconstruction.

On December 20th, 1906, reconstruction on the existing levees was begun by tearing away the land side of the levee and excavating a

continuous muck-ditch as deep as the test-pits indicated to be necessary. The usual location of a muck-ditch is under the center of the levee section, but, in reconstruction work, this was not practicable, and besides, there were several reasons for location nearer the land toe, which will be mentioned later. The excavation was made as narrow as possible with the use of 4-horse Fresno scrapers, and it was found that the walls, not only of the natural soil, but of the recently constructed levee section, stood practically vertical without any caving. (Fig. 43.) The muck-ditch was excavated 1 ft. lower than the lowest layer of cracked adobe soil lying above the permanent water-table, and was refilled with the material removed, care being taken to keep out roots and clods of adobe exceeding 3 or 4 in. in greatest dimension. When the muck-ditch was completed, the land side of the levee was replaced to the slope of 3 to 1, instead of 2 to 1, as originally built. This work was started on force account, but was soon changed to a yardage basis, on the following schedule:

Levee section removed and replaced in embankment, 12 cents per cu. yd.
Muck-ditch excavation and refill, 17½ cents per cu. yd.
Reinforcing levee section, 19 cents per cu. yd.
The total earth handled was 199 000 cu. yd.

Levee Extension.—At the same time, surveys were commenced for extensions of the levee to the south. From Mile 7 the original alignment continued closely paralleling the river, and here all clearing had been done for 2 miles, and about 20% of the fill had been made. At the south end the flood caused three breaks, close together, through almost completed levee section. These breaks were due to bad material, that is, adobe which in working had broken up into small, hard clods. Directly across the river, similar breaks had occurred in the levee of the Yuma Project. In addition, the river during this flood showed a marked tendency to cut into the west bank immediately opposite.

The surveying party found that the most suitable soil and the highest ground, west of the river bank itself, lay along the Paredones overflow channel, which turned away more than a safe distance from the river. This channel was followed beyond the undergrowth to open country, and a hurried reconnaissance showed that at some future time, by building approximately 20 miles of levee, most of it relatively low,

connection could be made with the mountain chain on the west side of the delta at Cerro Prieto.

It was felt that the work now in progress was not merely to prevent flood-waters from quickly getting around the end of the Hind-Clarke Dam, but that very soon a system of dikes constituting a continuous line of defence for the Imperial Valley must be provided. President Roosevelt had not requested or authorized Mr. Harriman to construct anything so elaborate, but only what would form a protection for a few years at most, or until suitable permanent arrangements could be made to control the river properly. Such levees as were to be built, however, obviously should be constructed with a view to being incorporated in their entirety in the final scheme.

Two plans for keeping the overflow waters of the Colorado from getting into the Salton Sea are at once seen to be better than any others. One is a levee line along the Paredones ridge and north of Volcano Lake to Cerro Prieto, and the other is a line of levees parallel to the river practically to tide-water. These two plans are shown in Fig. 4. The first would protect all American interests and all that territory in Mexico lying to the north and west; the second would also protect the very large area lying between. This additional area is the property of the C. M. Co., and is in an extremely good location for irrigation from the Colorado. The C. M. Co., on being approached regarding the matter, thought that its present interests would be best served by the northern line of defence, which thus precluded the possibility of financial assistance from that company in giving additional protection, and the matter resolved itself into a decision as to which of the two lines could be built and maintained at least cost. The instrumental data regarding the region were insufficient to decide between the two possibilities, and, incidentally, the same condition of affairs unfortunately exists to-day. However, something had to be done, and at once. Based solely on careful reconnaissance, the northern line of levees was decided on, and the portion to be immediately built was located to a point about 10 miles below the Hind-Clarke Dam, this being enough to prevent water from getting around into the Alamo channel during flood stages. Aside from the desire to do only what was absolutely required and could certainly be finished before the summer flood, it was felt that the officials of the Mexican and American

Governments should have the greatest freedom in determining on future work for controlling the river.

The grade was determined from existing high-water marks along the line of the levee, checked by high-water marks along the line of the river; and several miles at the lower end has a grade of 3 ft. per mile. The cross-section decided on was a top width of 10 ft. and slopes of 3 to 1 on each side.

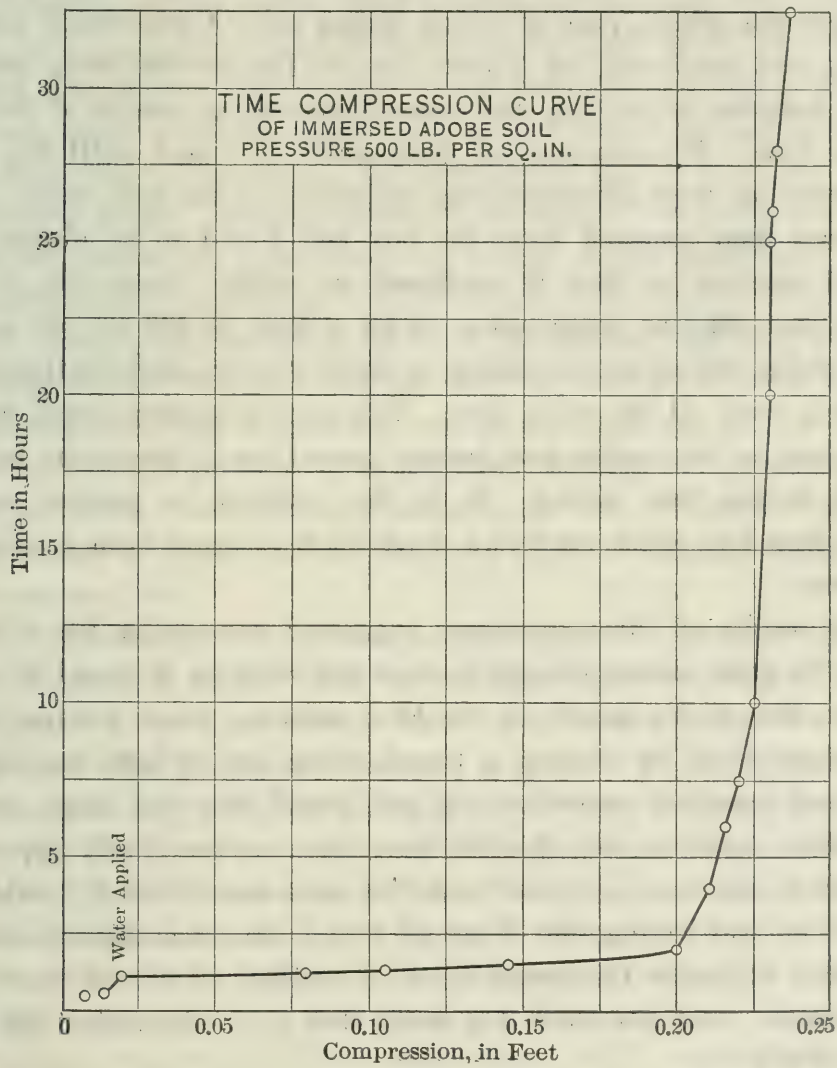


FIG. 49.

Muck-Ditches.—The Yuma Project, U. S. Reclamation Service, loaned one of its engineering corps, the method being to grant leave of absence to the members and have them carried on the payrolls of the Mexican Co. This corps was in charge of Mr. C. W. Ozias, Assistant Engineer, and was assigned to levee location work. Mr. Ozias was

instructed to make tests for determining the time at which efficient compression of adobe soil would take place with distributed loads, as under a levee when immersed in water. For these tests some hard chunks of adobe were taken from the muck-ditch excavation, 2 ft. below the surface at Station 178, and placed in a box 1 ft. in each dimension, in as nearly natural condition as possible, and kept under a continuous pressure of 500 lb. per sq. ft. At the end of 1 hour ½ cu. ft. of water was added, part of which leaked out. Compression started rapidly, and continued for 1 hour, due to the chunks being pressed closely together in the box, and slower compression started at the end of this time. The compression continually decreased until the 40th hour, when no more movement was noticeable on the scale beam. The mass was then removed from the box and found to be plastic and pressed together so that it contained no voids. From this it appeared that efficient compression under a load of 500 lb. per sq. ft. starts at the end of an immersion of about 3 hours, and continues indefinitely until all the voids close. This is why borders, canal banks, levees, etc., in that region soon become impervious if breaks can be prevented during that period. It is the inability to prevent breaks over a long line which renders a muck-ditch of some form practically necessary.

The results of this experiment suggested dampening the slickens under the levee section enough to have the voids in it closed by compression due to the weight of the levee section. Such wetting could be brought about by digging a trench along one or both toes of the levee and pumping water into it, and would save the large cost of muck-ditch construction. As this levee line was too vitally important for such an innovation, it was decided to use a muck-ditch 6 ft. wide on the bottom and having side slopes of 2 to 1, the excavation to extend at least 1 ft. below the lowest layer of cracked adobe soil above the water-table. Test-pits were dug every 200 ft. to determine the necessary depth.

The muck-ditch, however, was located under the land toe of the levee, as shown in Fig. 38, because it was not deemed desirable to prevent water from getting under the levee, but, on the other hand, to allow water to go under it as far as possible and still certainly hold it back with the muck-ditch. In this way the cracked adobe layers under the

levee section would become impervious as a whole in the minimum time after the water came as high as the toe of the levee.

The muck-ditch, except for two short stretches, was filled with the soil taken from it, only a few places being found in the 17 miles constructed where bad material occurred in such masses as to render it at all difficult to mix the adobe clods with satisfactory material taken out of the excavation.

Borrow-Pits.—Borrow-pits were located on the land side, leaving a berm of 40 ft. and traverses 20 ft. wide at intervals of 250 ft. It was specified that borrow-pits be left in workmanlike condition, with a maximum depth of 2½ ft. on the side nearest the levees and 4 ft. on the farther side. The reasons for borrow-pits on the land side have already been given. The occurrence of the second break was not considered to indicate any good reason for change in this particular.

Contracts and Contractors.—All levee extension work was done on a yardage basis, a contract being let to W. K. Peasley and Company, of Los Angeles, for 18½ cents per cu. yd., for the entire levee cross-section, including the muck-ditch, and 10 cents per cu. yd. for refilling the muck-ditch section. The price for clearing and grubbing was $2 000 per mile. The contractor assumed all risk of expenses due to delays caused by flood, and the company paid all customs charges at the Boundary Line.

The Yuma Project, U. S. Reclamation Service, permitted the use of 120 head of its rented stock in order to assist in completing the work before the summer flood. This outfit worked under the usual regulations of the Reclamation Service, on a rental basis, and the levee contractor put it on the work and paid the Mexican Co. what the latter paid. The Reclamation Service engineers advise that levee construction on the Yuma Project has been found to cost much less than Mr. Peasley's contract, but, for some reason or other, the levee contractor and the company's timekeepers, who checked the work, show that the contractor's price for the work done by this grading outfit on this job was less than its cost. The great hurry had a marked effect on the expense to the contractor, however. At any rate, the writer had an opportunity to examine the contractor's books later, and found that his profit on this job, with proper overhead and equipment deterioration charges, was 17.3%, and none of the possible delays due to floods, etc., occurred.

All work was very carefully inspected by an unusually large corps of men, and every precaution was taken to prevent any roots or rubbish from getting into muck-ditches or fills, and to see that all muck-ditch construction was carried to the depths ordered.

Railroad Track.—When completed, all levees, both as reconstructed and extended, were laid with a railroad track consisting of new 6 by 8-in. Oregon pine ties 8 ft. long and good old 56-lb. relaying steel obtained from the Southern Pacific Company. This was done partly because of the great advantage in maintenance work in future and partly to distribute a blanketing of Mammoth pit gravel.

On the completed work 15 miles of track were laid on the main levee, 1½ miles on the spur levee, 5¾ miles from Hanlon's Junction to the Lower Heading, 2.6 miles of sidings, quarry tracks, etc., in California, and 2.7 miles of sidings and double track over the Hind-Clarke Dam in Mexico. No part of this track has been taken up, but the main line and sidings between Hanlon's Junction and the end of the spur levee have been sold to the Inter-California Railroad.

Gravel Blanketing.—Vegetation, springing up like mushrooms on fills in the region, very soon precludes any possibility of inspection. The roots also attract burrowing animals. Furthermore, the danger of erosion on the water face by swift currents due to the fall of 3 ft. per mile made it exceedingly desirable to provide a better surface for the eddy currents. For these reasons all levees were blanketed on the top and both sides with a 15-in. layer of gravel from the Mammoth gravel pit, which, as has been said, supplies a cementing material which packs into an almost impermeable surface. It was practically impossible—and would have cost more money—to have blanketed the water face only, or to have put a greater depth on it than on the land side, because all the gravel was hauled in "battleships" which dumped equal quantities of material on each side. Ordinarily, two cuts of cars were unloaded, a considerable portion of the first being used on the top and to surface the track. The remainder was spread evenly over the two slopes of the levee with a home-made spreader devised and constructed with an ordinary flatcar temporarily taken for the purpose by Superintendent Hind at a total cost of $300. Its construction is shown by Figs. 45 and 52. With this the gravel was spread in an extraordinarily workmanlike manner at a cost of 0.1 cent per cu. yd. It worked in either direction, and the ordinary process was to have one cut of gravel

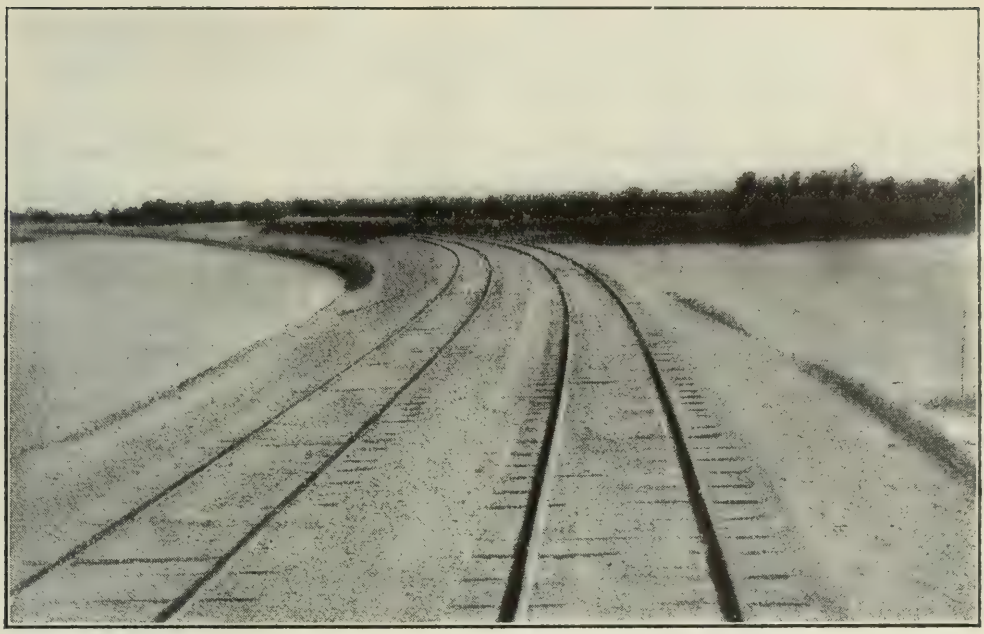


FIG. 50.—SOUTH END OF HIND-CLARKE DAM, SHOWING LOCATION OF SECOND CLOSING.

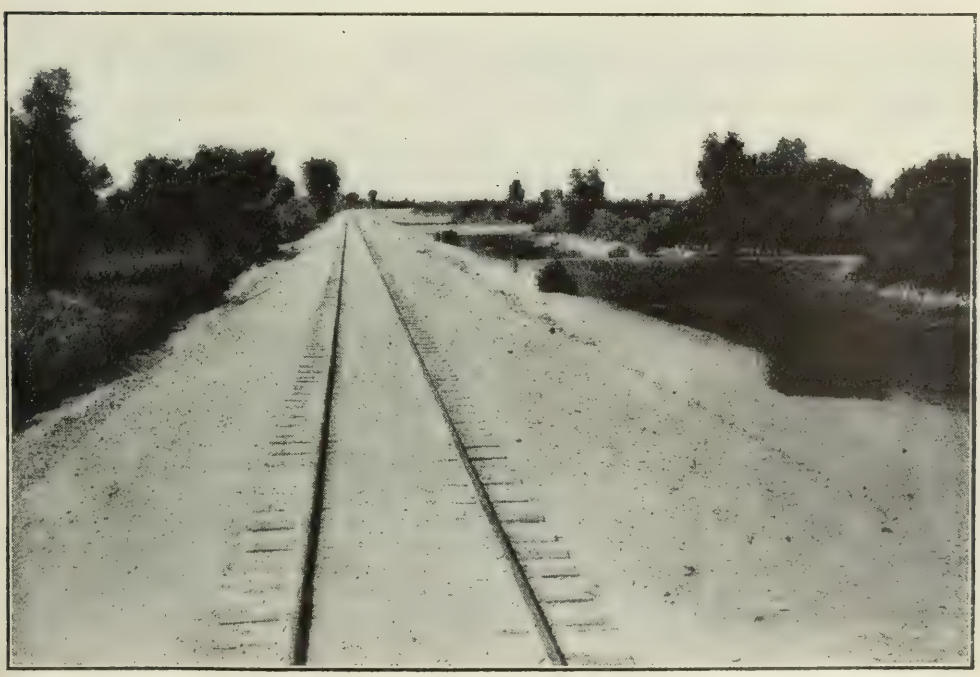


FIG. 51.—TYPICAL SECTION OF COMPLETED LEVEE WITH LAND-SIDE BORROW-PITS.

1286.

dumped along from 3 000 to 6 000 ft. of levee, and have a locomotive take the spreader on two round trips, the usual speed being from 10 to 12 miles per hour.

Very much more permanent and expensive blanketing of levees is to be found on the Sacramento River, California. In Reclamation District 307, near Lisbon, about 15 miles below Sacramento, a length of a little less than 3 miles of levee has recently been completed. This keeps out the back waters, in which there is little current, but, on account of the width, the wave action is relatively severe. The blanketing was begun about July 1st, and finished about December 1st, 1911, and consists of 700 000 sq. ft. of reinforced concrete 4 in. thick. (Fig. 56.) The reinforcement was a No. 10 gauge wire, 6 by 6-in. mesh, known as the Clinton Electrically Welded Fabric. The concrete was a 1:2:5 mixture, with rock brought by train from Oroville to Sacramento and thence by barge to the work, and cost there about $1.50 per cu. yd. The Reclamation District furnished the cement and reinforcing material, and the contractors, Richard Keating and Sons, of San Francisco, did all the other work. Mr. P. N. Ashley, County Surveyor of Yolo County, Woodland, was engineer. This covering cost 13 cents per sq. ft., or a total of about $94 000, or approximately $32 000 per mile. The average height of the levee was 22 ft.

TABLE 16.—Statement of Charges on Account of Operation of Mammoth Gravel Pit, from October 15th, 1906, to July 15th, 1907, Inclusive.

Labor of gangs in pit getting out gravel		$23 312.46
Use of tools, 2% on above labor		466.25
Wages of trainmen in pit		3 108.91
Wages of enginemen in pit		2 111.65
Wages of operators, etc., at pit		897.19
Rental of engine in pit		2 202.50
Rental of steam shovel in pit		2 457.50
Fuel furnished for pit engine		4 495.14
Fuel furnished for steam shovel		1 594.41
Material purchased for pit engine repair	$184.63	
Material purchased for steam shovel	548.90	
Miscellaneous supplies	6 890.88	
Miscellaneous pit engine supplies	237.98	
		7 862.39
On store department expense, 5%		393.10
Shop repairs to steam shovel		727.19
On shop expense, 10%		72.72
Freight on material shipped to pit		6 345.86
Miscellaneous credit		*119.74*
Total		$55 927.53

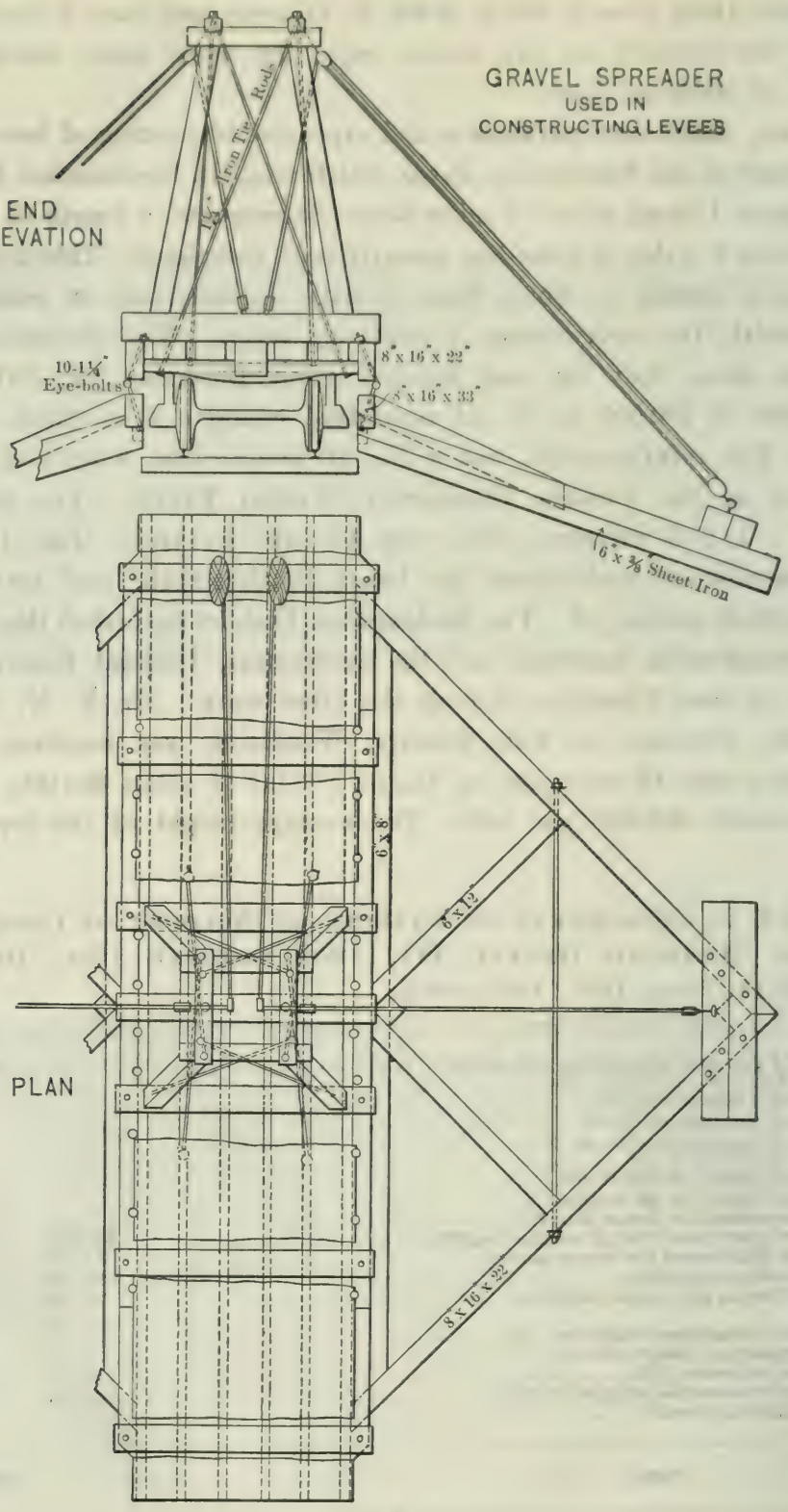

GRAVEL SPREADER
USED IN
CONSTRUCTING LEVEES
END
ELEVATION
Iron Tie Rods
10-1¼" Eye-bolts
8"x 16"x 22"
8"x 16"x 33"
6"x ⅜" Sheet Iron
PLAN
6"x 8"
6"x 12"
8"x 16"x 22"

FIG 52.

TABLE 17.—Carloads of Gravel Shipped.

Month.	To California Development Company.	To Epes Randolph, Agent, S. P. Co.	To Southern Pacific Company.	Totals.
1906.				
October	357	.....	....	357
November	1 171	.....	18	1 189
December	.....	593	718	1 311
1907.				
January	.....	479	644	1 123
February	.....	756	726	1 482
March	.....	1 849	300	2 149
April	.....	2 178	114	2 292
May	.....	2 032	233	2 263
June	.....	1 082	527	1 609
July	.....	239	59	298
Total	1 528	9 206	3 339	14 073

The following is a detail of charges for royalty to Epes Randolph, Agent, S. P. Co.:

Value of tracks in Mammoth gravel pit, property of S. P. Co. $16 832.00

Interest on $16 832.00 for 1 month at 6% per annum $75.74

Depreciation on $16 832.00 for 1 month at 5% per annum 70.13

$145.87

Interest from December 1st, 1906, to July 15th, 1907, 7½ months 1 094.03

Total number of cars removed by Epes Randolph, Agent, S. P. Co., 9 206, or ..... $0.11883 interest per car.

Charge for gravel, 1 cent per cu. yd., 30 cu. yd. per car, or ..... 0.30 royalty per car.

Total royalty charged per car $0.41883

Average cost per car, $55 927.53 divided by 14 073 = $3.9741014 per car.

In blanketing the levees, 5 285 carloads, or 185 000 cu. yd., were used. Of this, 4 803 carloads, or 168 000 cu. yd., were used on the 15 miles of main levees, or 16 800 cu. yd. per mile, and 482 carloads, or

17 000 cu. yd., on the 1.6 miles of spur levee, or 10 633 cu. yd. per mile.

Wing Levees.—In addition to blanketing the levees with cementing gravel, wing levees of gravel were built out into the brush, at intervals of about 400 ft. Their purpose is to check the flow of water in the V-shaped section between the trees along the near toe and the water face of the levee. Brush abatis work was not considered sufficiently permanent.

Telephone Line.—A two-wire, metallic-circuit, telephone line was constructed along the land toe of the levee after the gravel blanketing was finished. The location was unfortunate, as the rapid growth of willows and other bulb vegetation just behind the levee caused considerable annoyance and expense. It would have been much better to have put the line on the land slope, about 9 ft. from the center of the levee.

At each mile post a 6 by 8-ft. wooden tool and telephone house was put up, and white boards were nailed to telephone poles at points half way between, to mark the ends of the patrol beats. When floods are expected, sacks, tools, etc., can be distributed and kept in these houses, but between times it was found impracticable to leave anything but the telephones on account of malicious and thieving passers-by. It was also found that the heat in these houses during the summer was sufficient to melt down and spoil many of the rubber receivers, so that only metallic ones are now used.

General Data.—The total length when complete was 15 miles of main levee, having an average height of 8 ft. and an average muck-ditch yardage of from one-third to one-fourth of the main section, and 1.6 miles of spur levee, with a height of from 20 to 4 ft. and a muck-ditch yardage of about 15% of the main section. The quantities and costs of the extension work after the second break were as follows:

154 500 cu. yd. muck-ditch excavation at 18½ cents.
49 700 " " " " refilling, which was twice handled, at 10 cents.
443 000 " " embankment at from 18½ to 25¾ cents.
166 700 " " original specification at 18½ cents.
100 800 " " wide check at 19 cents.
154 545 " " checkerboard system at 25¾ cents.
9.66 miles of clearing and grubbing, $2 000.

FIG. 53.—TYPICAL SECTION OF COMPLETED LEVEE, WITH INTERRUPTED BORROW-PITS ON THE RIVER SIDE.

FIG. 54.—CONNECTING CHANNEL ERODED BETWEEN INTERRUPTED BORROW-PITS DURING SUMMER FLOOD, 1907.

Enlarging Main Canal and Building Secondary Levee on Canal Bank.—On November 26th, 1906, the *Delta* cut its way into the Main Canal just below the Boundary Line and commenced deepening it and building a secondary levee on its river side beginning at the concrete head-gate and working down stream. It reached the Lower Heading and connected the levee bank with the north end of the Hind Dam in May, 1907, and started back, improving the work at various places and continuing this until July 21st, 1907. This levee was built in exactly the same way as those around the Reclamation Districts of the Sacramento River, and was made with a minimum height of 3 ft. greater than the main levee opposite along the river bank. Though not a very efficient construction, it, nevertheless, will serve to prevent water from getting into the Main Canal from breaks in the main levee until they can be repaired.

Criticism of Levee Work Done.—All parties interested assumed that the Harriman interests were doing work in the capacity of a contractor for the United States Government, and not in any sense in its own behalf, and that the engineers of the Reclamation Service were "available for consultation." Certain of these engineers, individually, criticized the excavation of the borrow-pits on the land side of the levee, and so a request was addressed to the U. S. Reclamation Service for a Consulting Board to consider the situation and make recommendations. Such a Board was appointed, all being members of the Service, consisting of A. P. Davis, W. H. Sanders, D. C. Henny, and Francis L. Sellew, Members, Am. Soc. C. E. This Board inspected the works then under way, and on January 10th reported, recommending among other things that:

"4. Borrow-pits should be on the water side, berms between them of greater width than the pits, and care should be taken not to disturb the vegetation on the berms, which should also be protected at frequent intervals with barbed-wire fences of 4 to 5 wires, the bottom of which is at the surface of the ground * * *.

"7. The levees now built should be provided with cut-off trenches under the water slope and later provided with sheet-piling reaching below borrow-pits * * *.

"8. All levees should be blanketed with gravel on water slope and railroad track maintained on the levee."

The clearing for half of the extension work was done at this time, and the work already started was continued as theretofore. Fear of

erosion trouble, because of borrow-pits on the river side, and correspondence with the New York office of the Harriman Lines, resulted in the appointment of another Consulting Board consisting of Messrs. L. C. Hill, W. H. Sanders, and F. L. Sellew, Members, Am. Soc. C. E., of the U. S. Reclamation Service, and William Hood, M. Am. Soc. C. E., Chief Engineer of the Southern Pacific Company, which met at Yuma, thoroughly examined the work, and reported on February 14th, among other things, as follows:

"Existing Levee Between Cement Head-Gate and Dam Across Lower Intake.

"Spur dikes (traverses between borrow-pits) to be increased in width to at least 50 ft. on top and to be at least 4 ft. in height above the general level of the original surface of the ground, and to extend at least 300 ft. northerly from the northerly edge of the borrow-pits, and in this 300 ft. no borrow to be made on either side of this levee and no brush to be cut outside of the limits of the slope stakes.

"The end of the levee and for some distance on the sides on each side near the end to be thoroughly brushed.

"These cross-dikes to be not to exceed 600 ft. apart, and where now located essentially farther than this, an intermediate cross-dike to be put in * * *.

"An abatis work, being in effect a wire and brush wing dam, shall be built from a point on the slope of the levee nearest the river, situated well above high water, and such wing dam pointing down stream, approximately, per local conditions, at an angle of 45 degrees to the levee.

"This to be made with suitable posts or stakes driven into the levee and between the levee and existing trees and thence by assistance of trees acting as posts or stakes and suitable barbed wire fencing in two lines not less than 2 ft. apart, and the whole filled with brush thoroughly wired down.

"These wing dams to occur at no greater distance apart than 500 ft. * * *.

"Levee Below Dam Across the Lower Intake, This Being Partly Completed and Under Construction.

"The same remarks as to spur dikes and abatis work apply as stated above with reference to existing levee between the cement head-gate and the dam across the lower intake * * *.

"For the levees as constructed, material has been taken from the land side instead of the river side, as recommended by the previous Consulting Board of the Reclamation Service.

"As these conditions now exist, the present recommendations are with a view to make the levees as secure as practicable under present conditions * * *. As to the still unconstructed portion of the 10¼ miles of levee now intended to be constructed southwesterly from the dam across the lower or Mexican intake, this unconstructed portion being several miles, we recommend and expect that the recommendations of the Consulting Board of the Reclamation Service as to the position of the borrow-pits, position, depth and character of muck-ditches, and all other matters, be strictly complied with."

Accordingly, the additional levees, constituting 4.11 miles at the south end, were built with muck-ditch under the river slope and with borrow-pits on the water side of the levee. These pits are 100 ft. lengthwise of the levee, with spaces between them 100 ft. in width, on which every care was taken not to disturb the vegetation to the water toe of the levee. This method of borrow-pits is known locally as the "checkerboard" system. (Fig. 39.)

It is obvious that the checkerboard system requires an average haul of excavated material of about 175 ft. in excess of that required by the ordinary plan.

According to comparative tests, this increase in length of haul and inconvenience in handling teams resulted in increasing the cost of work about 6¾ cents per cu. yd. for embankment only, or approximately 30% of the cost of team work.

It was deemed advisable to complete the main levee high enough to be beyond danger of overtopping by floods before doing anything on the spur levees between the land side borrow-pits recommended by the Consulting Board on February 14th. About the middle of May bids were asked for constructing them (described in the first item of the report of the second Consulting Board), and the lowest bid was $40 per acre for clearing, $80 per acre for grubbing, and 31 cents per cu. yd. for embankment, making their estimated cost between $12 000 and $12 500 per mile of main levee. Accordingly, a third Consulting Board was requested, and was appointed by Mr. A. P. Davis, Chief Engineer of the Reclamation Service, consisting of C. E. Grunsky, Consulting Engineer to the Secretary of the Interior; L. C. Hill, and F. L. Sellew, which Board met on June 19th and recommended as follows:

"1. That in lieu of the general system of cross-dikes recommended by the second Consulting Board:

"(a) To complete at once the cross-dikes now being constructed

about 500 ft. southerly from the south end of the Clarke dam, making it 10 ft. wide on the top and giving it the same height as the crest of the spur levee with which it connects;

"(b) To construct a second cross-dike about 500 or 600 ft. southerly from the first, also 10 ft. wide on top and with a height the same as the crest of the spur levee with which it connects;

"(c) To construct cross-levees, with crest at least 4 ft. above made ground (tops 10 ft. wide) between the main levee and the secondary levee [along the east bank of the main canal from the concrete head-gate to the north end of the Hind Dam. H. T. C.], near or on the southerly bank of the old upper Mexican intake, and a second within about 1 000 ft. of the southern end of the secondary levee."

This recommendation was carried out.

Summer Flood of 1907.

The levees, including the secondary one along the Main Canal and the cross-levees mentioned in the recommendations of the last Consulting Board of the Reclamation Service, were completed on July 21st, 1907, and the Epes Randolph, Agent, fund was closed. The *Delta* continued deepening the Main Canal and at the same time raising and strengthening the secondary levee on its river side for 3 months longer, but it was estimated that the cost of such work balanced the deepening of the Main Canal done while strengthening the same secondary levee, prior to that date. After that time all charges for maintenance and operation of the head-gates and levee system were made by the Mexican Co. against the C. D. Co.

The flood of 1907 was a record one in total discharge, and probably would have been in gauge height reached at Yuma had it not been for the unusual conditions lower on the river. The old channel of the Colorado was considerably higher than usual, from the point of diversion to the Gulf, the erosion since the diversion and until the coming of the summer flood being of not much importance. It had been silted by the very small quantity of water carried as the re-diversion became more and more a reality, and, in addition, it was appreciably raised by the flood of December 5th which stranded quantities of heavy silt.

The vegetation of the lower delta lands depends on the annual overbank flows, and these lands had not been covered for two seasons, due to the river diversion north of the delta's dividing ridge, much of the light vegetation had perished, and large tracts of the region had been burned over. In designing the levee system it was deemed conservative

practice not to take into account any such increased overbank flow, and to consider that the whole channel would be much less efficient and would deepen much more slowly than usual under the coming summer flood, on account of the fact that the bed had been undisturbed for 2 years, and hence was compacted and dry. Cross-sections are shown on Plate L. However, the flood of 1907 came up very gradually, and eroded the bed most satisfactorily, and an extraordinary quantity of water went overbank, particularly to the west because of the greatly decreased vegetation. It thus happened that this record flood, confined between levee banks only 1 500 ft. apart, rose to only an average of about 6½ ft. below the top of the levee, varying from a minimum of 6 ft. to a maximum of 7 ft. The water got against the levees for their whole length, however, testing the muck-ditch construction thoroughly, and in no case was any weakness apparent.

The water-table throughout the region was raised above the bottom of the borrow-pits in many places, so that a considerable quantity of water, in some cases 2½ ft. in depth, slowly seeped into these pits. This water was always perfectly clear, came in very slowly, and gauge readings, kept in many of them and in pits nearby, showed that the levels in them fluctuated with the adjoining water-table at all times. As the flood went down, these waters lowered and disappeared, and a rapid growth of willows started, so that a very large part of the borrow-pits is now overgrown with a dense growth, many bushes attaining a height of 20 ft. in 2 years.

Much difficulty has been found in keeping Indians from clearing away such growths and utilizing the borrow-pits as garden patches. Indeed, the Mexican officials practically take the position that, though the land is private property belonging either to the Mexican Co. or the C. M. Co., the nomadic Indian tribes have for many years been free to live their Gipsy life therein; that the levee system has cut off the annual inundation on which these people depended for their garden crops; and that it is unreasonable to insist on preventing them from taking these borrow-pits for gardening purposes, particularly as they are prevented from making clearings and utilizing inundated land in front of the levee.

A very striking occurrence was the complete filling up of the diversion channel between the river and the Hind-Clarke Dam. In many places this was more than 45 ft. in depth and probably averaged

20 ft. for 2 800 ft. up and down the river and 3 300 ft. at right angles thereto, an area of 210 acres. When this summer flood had passed, there was only a little pool about 5 ft. deep immediately above the Hind-Clarke Dam where the water had been deepest. Two years later this area was so densely overgrown with willows that it was extremely difficult to believe that the break ever occurred there. Until this silting up in front of the Hind-Clarke Dam began, numerous small boils appeared in the sand bar formation behind the Hind Dam. These at no time were of any importance, and soon ceased. Behind the Clarke Dam, practically no seepage whatever occurred.

The very slight percolation of water under these structures was rather surprising and very gratifying, considering their non-homogeneous structure and the character of their foundation. Nevertheless, it accords with the experience with the wooden and concrete head-gate coffer-dams, and the pumping of water into the Main Canal to keep the *Alpha* afloat, namely, that seepage through the alluvial soil of the region is remarkably small.

The water-table, for considerable distances on each side of the river, rises and falls with the water surface, especially during the summer floods, with small lag in time. The writer has always considered these two facts incongruous, and has never been able to find a satisfactory explanation.

Maintenance of Levees.

Until the summer flood of 1907 had passed, the levees were very carefully patrolled, watchmen being stationed night and day at each telephone house. A considerable store of timber, large and small, has always been kept at the Lower Heading until recently, and a storehouse containing a large quantity of shovels, picks, crow-bars, track tools, and lanterns has been maintained at the upper end of the Hind Dam. In this, from 20 000 to 50 000 sacks are always kept, being drawn on for use in the valley and replenished to avoid deterioration from long storage. Until a number of flashy floods had passed, a work train was always ordered from the Southern Pacific on reports from the head-waters indicating large rises in the lower river, and sacks were taken from the storehouse and distributed up and down the river, 500 at each toolhouse. Confidence in the effectiveness of the dikes soon grew, so that this is no longer being done.

FIG. 55.—SECOND CLOSING. HEAD DEVELOPED, 8 FEET.

FIG. 56.—REINFORCED CONCRETE FACING, ON LEVEE 25 FEET HIGH, NEAR SACRAMENTO, CAL.

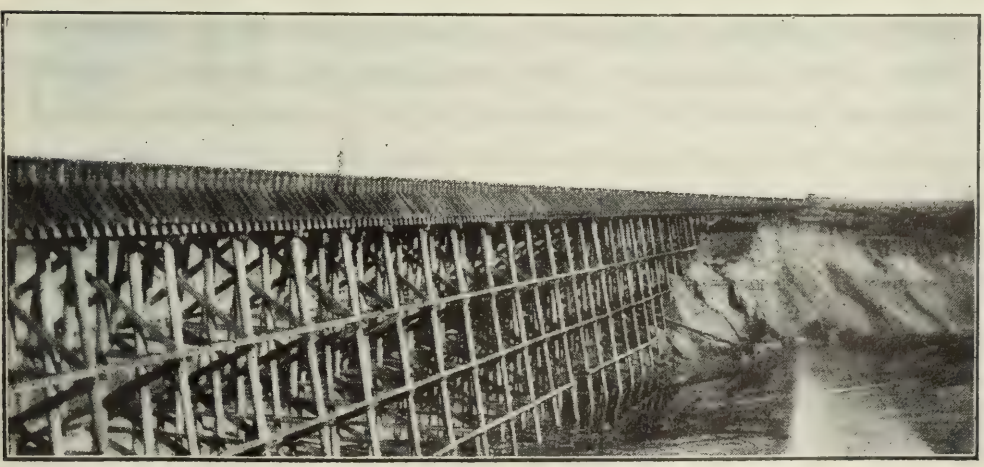


FIG. 57.—WOODEN FLUME OVER NEW RIVER BARRANCA.

In spite of the gravel blanketing, vegetation starts up slowly, and is kept down by constant but relatively inexpensive work. Considerable annoyance is caused by insects, particularly large burrowing ants which make great ant-hills and holes. These are destroyed by pouring gasoline into the holes and burning it.

As the value of rock, in repairing breaks or in river protection work, to prevent the side cutting of banks from breaching the levees, made it advisable to have a large quantity ready in the Andrade quarry, about 50 000 cu. yd. were blown out ready for handling by steam shovels. This was done by "coyoting" or driving 3-ft. tunnels horizontally about 30 ft. into the foot of the rock face, at intervals of 60 ft., and driving cross-tunnels at the ends, 10 ft. on each side. These cross-tunnels were loaded with black powder, which was all exploded at one time. All this rock has been taken away, and has left the quarry with a practically vertical face, averaging 50 ft. in height for a length of 1 000 ft., a very satisfactory condition. An American Hoist Company's 10-ton steel derrick with a 60-ft. boom, erected to handle this rock, proved very efficient and satisfactory. The railroad north of Andrade was raised above flood heights, the quarry lines were thrown over, and a track was laid on the top of the concrete head-gate, stringers being laid on the cellular pier walls. This track was connected with that on the levee, so that a train could be loaded at the quarry and run over the concrete head-gate and down the levee system, using the main line of the Inter-California Railroad only for that portion lying on the Hind-Clarke Dam. In the fall of 1907 this railroad was reconstructed and repaired from Calexico to Cocopah, and extended through to the end of the spur levee. The track from this point to Hanlon's Junction was taken over by the Southern Pacific Company, in accordance with arrangements made when the branch line was built. For this, consisting of 6.1 miles of main line and 2.1 miles of siding, the Inter-California paid the C. D. Co. approximately $65 000. When this was done the branch line was no longer available for use by the irrigation company, but the right to use the east track or siding on the Hind-Clarke Dam was retained, and the Inter-California was specifically released from any obligation to maintain the dam.

It certainly is not often that a diversion head-gate carries a main-line, standard-gauge railroad, and the fact that this does will give

some idea of its size. By thus utilizing it, the heaviest carloads of rock from the Andrade quarry can be hauled over the levee system, and be independent of the Inter-California tracks.

The Southern Pacific Not Reimbursed by the United States Government.

Early in 1907, a bill* was introduced before Congress:

"To reimburse the Southern Pacific Company the amounts expended by it from December first, nineteen hundred and six, to November thirtieth, nineteen hundred and seven, in closing and controlling the break in the Colorado River.

"Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That the sum of one million six hundred and sixty-three thousand one hundred and thirty-six dollars and forty cents is hereby appropriated, out of any money in the Treasury not otherwise appropriated, to reimburse and pay the Southern Pacific Company the amounts paid by it from December first, nineteen hundred and six, to November thirtieth, nineteen hundred and seven, in closing and controlling the break in the Colorado River and thereby saving the overflow and destruction of the Imperial Valley in southern California."

This bill was referred to the Committee on Claims, of which Hon. James M. Miller, of Kansas, was Chairman, and consideration was begun on February 24th, 1908. At this hearing Mr. Maxwell Evarts, Counsel of the Harriman Lines, Mr. P. G. Williams, of the Accounting Department, who had been in charge of the accounts of the Epes Randolph, Agent, fund, and the writer, appeared to make explanations and present for examination vouchers covering all items of expense. President Roosevelt was frank and open in urging the justice of the claim, but, of course, was not advised as to whether such amount of money had been expended. Everyone else, however, seemed to be very much afraid of the matter. The Committee on Claims desired information as to the character of the work done and the fairness of the charges therefor, and asked the Secretary of the Interior, James R. Garfield, to have the Reclamation Service Engineers investigate and make a report. The reply was that no funds were available for that particular kind of work. Accordingly, President Roosevelt asked Secretary Garfield, F. H. Newell, M. Am. Soc. C. E., Director of the

* H. R. 13 997, 60th Congress, 1st Session.

Reclamation Service, and Mr. Walcott, formerly head of that Service, to take up the matter and report on what the Government, as a matter of moral and equitable obligation, should pay for the service rendered. President Roosevelt at the time, March 11th, in a letter to the Chairman of the Committee on Claims, advised him that this had been done; recalled the dire need for immediate action at the time; stated that negotiations with Mexico were then under way for future action in concert between the two nations; that he did not think the Southern Pacific people should be obliged to wait for the conclusion of these negotiations between Mexico and the United States with regard to future action, but that a rough estimate should be made as to what the United States should pay as reimbursement to the Southern Pacific Railroad, and that it was an act of justice to deal generously in the matter, for the railroad, by its prompt and effective work, rendered to a threatened community a notable service which could in no other way have been done. To facilitate matters, Mr. Evarts agreed with the Committee that Mr. Grunsky, who had been Consulting Engineer of the Secretary of the Interior on all Reclamation Service matters until a few months previous, and who, of all Government engineers, knew most about the Lower Colorado River and the operations of the Southern Pacific Company, should be engaged by the Committee to report on the work done and the expenditures, and that Mr. Evarts would advance the expenses of such investigation and report.

Mr. Grunsky, with the assistance of the American Audit Company, of New York City, investigated the accounts, and reported on April 1st that the structures along the river were adequate and efficient, and fulfilled their intended purposes; that additional protective work would have to be done in the near future; that charges incurred prior to December 7th, 1906, and subsequent to July 21st, 1907, were not properly chargeable to work done as a result of correspondence between the President and Mr. Harriman; that the quantities of material, such as rock, gravel, and earth, covered by the expenditures could not then be ascertained by measurement of completed structures with any degree of precision; that the records presented by the company showing the quantity of rock, gravel, and clay put into the work from day to day from January 27th to July 18th, 1907, during which time practically all the work of closing the second break and com-

pleting the protection work was done, constituted a reasonable check on the bills rendered, and that the number of carloads of material handled prior to January 27th, 1907, was no doubt correctly reported; that a fair basis for all charges, everything considered, would be cost plus the usual 10% for superintendence, tools, etc.; that this basis had been followed in the accounts, with the exception of the freight, which had been billed at tariff rates and really should have been made at 0.5 cent per ton per mile; that the accounts revised and corrected according to the foregoing showed the total net expenditures to be $1 083 673.97, exclusive of interest.

TABLE 18.—EXPENDITURES ON THE COLORADO RIVER WORK SUBSEQUENT TO DECEMBER 1ST, 1906.

Labor	$275 310.12
Materials and supplies	261 969.04
Fuel	33 339.58
Freight charges on supplies and materials	613 150.84
Freight fuel	19 073.00
Transportation	12 395.23
Work-train service	7 627.17
Rental of equipment	70 507.54
Commissary supplies and labor	8 356.27
Trackage	31 981.00
Construction of additional levees (contract)	255 378.55
Officers' and clerks' salaries	8 449.08
Office expenses	1 279.20
Traveling expenses	1 437.65
Sundry expenses	1 091.39
Duties	34 717.45
Total	$1 636 063.11

Mr. Grunsky's report was clean-cut and fair, the inclusion of freight charges at tariff rates being done against the writer's advice. The Committee on Claims, therefore, now had definite ideas as to the work and cost. The report to President Roosevelt from Messrs. Garfield, Newell, and Walcott (dated March 17th), and mentioned in his letter of March 11th, as to the fair proportion to be repaid the railroad, was forwarded to the Committee, and, after reviewing the situation, including the exchange of telegrams between Mr. Harriman and President Roosevelt on December 19th and 20th, 1906, states:

"Under the circumstances, we do not feel justified in attempting even in a rough way to approximate the burden, other than to state that the principal beneficiaries are six in number: (1) The settlers in the Imperial Valley; (2) the Southern Pacific Co.; (3) the California Development Co.; (4) the Mexican Corporation; (5) the Republic of Mexico, and (6) the United States. Not considering the settlers in the valley, we have five distinct entities among whom the

burden might be distributed more or less equally. Thus, a rough estimate might apportion to the United States 20 per cent. of the money expended to reimburse the Southern Pacific Company for the actual expenditures of repairing the break in Mexico. Such proportion would fully comply with your suggestion that the United States Government should act generously toward the Southern Pacific Company, for by prompt and effective work it rendered a notable service to the threatened community of settlers in the Imperial Valley, quite regardless of the ultimate benefit of such action to the railroad company itself."

This recommendation has always seemed remarkable to the writer. The land interests in the Imperial Valley on both sides of the Boundary Line represent fully two-thirds of the present property values which had been threatened; the Southern Pacific Company about one-sixth; the United States, through its Laguna Weir, about one-sixth; the C. D. Co. and the Mexican Co. nothing, because they were both bankrupt, and the Republic of Mexico practically nothing because its interests conserved were wholly prospective, as well as those of the United States as far as irrigable land farther up the river is concerned. Furthermore, it was known that $950 000 had already been subscribed by the people and corporations in Imperial Valley when the President called on Mr. Harriman to start work, as he did, and at once stopped such subscription. It is obvious that the railroad could at this time in no possible way collect anything from any other source than the United States for such work, so that such payment as might be made by the United States will represent the grand total reimbursement. Just why this Committee eliminated the land owners and then considered the remaining five entities as being equally concerned has always seemed remarkable, especially because, when two years later another call for help from Imperial Valley came to President Taft, Congress at once appropriated $1 000 000 to protect the land owners primarily and almost exclusively.

The Committee took no action in the matter until Congress adjourned, and two years later, at the next session, another bill was presented, this time in the Senate (Senate 417, Sixty-first Congress). The matter was gone over again, and the bill with the amount cut to $773 647.25, or 71.4% of that reported by Mr. Grunsky, passed the Senate. The House Committee on Claims made a favorable report. Five members of the Committee submitted a minority report on January 28th, 1911, stating that they did not think there was any legal,

equitable, or moral obligation on the part of the Government to pay the railroad company any amount whatever for closing the break; that expenditures were made neither at the request of the Government nor for its benefit; and that the appropriation of such sum would be "purely a gratuity, a gift of the people's money to the Southern Pacific Railroad Company * * *.

"We oppose this proposed gift to the Southern Pacific Railroad Co. as well as all other gratuities to private enterprise."

The bill did not pass the House.

Damages Caused by the Runaway River.

The first damage caused by the diversion of the river was the flooding of the salt beds and the gradual burying of the entire plant of the New Liverpool Salt Company in the bed of the Salton Sea. The property was probably worth about $125 000.

As the waters continued to rise, they began to threaten the main line of the Southern Pacific Company throughout the basin, and in July, 1905, they reached the rails for a considerable distance. Shooflying was begun and continued from time to time, nothing very aggressive being attempted because of optimistic advices as to when the river would be under control.

Shooflies Nos. 1 to 7, inclusive, were at an elevation of —250 ft. or more. Shoofly No. 11 is 39 miles long, and follows the —200-ft. contour, being determined on when the probabilities of controlling the river before the summer flood of 1906 seemed to be rapidly decreasing. It was built in February and March, 1906, and was located with a view to being safe from the rising waters for at least two years, the estimates being based on the discharge records of the river at the time of building. As a matter of fact, some sections of the track a few feet below the —200-ft. contour were in trouble in the latter part of October, the water then being 47.5 ft. higher than when the line was surveyed. When the second break occurred, Shoofly No. 12, 48.9 miles long and following the —100-ft. contour, was hurriedly surveyed and graded during January and February, 1907, the outside drainage work being completed on April 1st. Track material for this had been gathered at each end—Mecca and Imperial Junction—during September and October, 1906, when Shoofly No. 11 was threatened. Practically none of the bridging was put in on this latter work, and

when only about 4 miles of track were laid it seemed that the river control work would prove effectual, and work was consequently ordered stopped.

The railroad company also suffered damage along the branch line between Imperial Junction and Calexico. At the crossing of the Alamo River north of Brawley the track was moved five times, the present alignment constituting a shoofly 2 706 ft. long, and introducing 105° of curvature, as compared with 16° 40′ originally. A few miles south, the enlargement of the New River channel made it necessary to construct three shooflies, the last one being 9 086 ft. long and containing 121° of curvature, as compared with 11° in the original alignment.

The total expenses incurred along the Salton Sea, exclusive of the cost of grading Shoofly No. 12, were as follows:

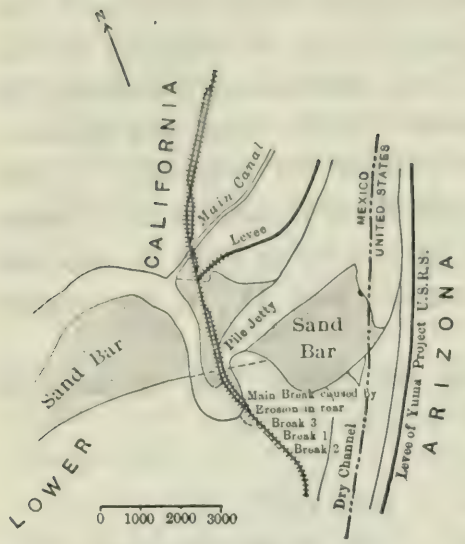
Year.	Labor.	Material.	Totals.	
1905	.....	.....	$148 183.71	
1906	$181 300.37	$307 763.58	489 063.95	
1907	49 875.96	37 678.08	87 554.04	
				$724 801.70

The damages sustained on the Imperial Branch were as follows:

Year.	Labor.	Material.	Totals.	
1905	.....	.....	$5 914.01	
1906	$19 222.37	$9 986.53	29 208.90	
1907	2 597.13	142.76	2 739.89	
				$37 862.80

The Inter-California Railroad, beginning at the International Boundary Line, was damaged more or less seriously for about 10 miles, the details being as follows:

Entirely rebuilding the road through Calexico and Mexicali and beyond, being cost less value of material recovered	$ 82 822.49	
Repairs to remainder of first 10 km., including water supply.....	4 259.76	
Repairs from 10-km. point to 14-km. point	21 163.73	
		$108 245.98

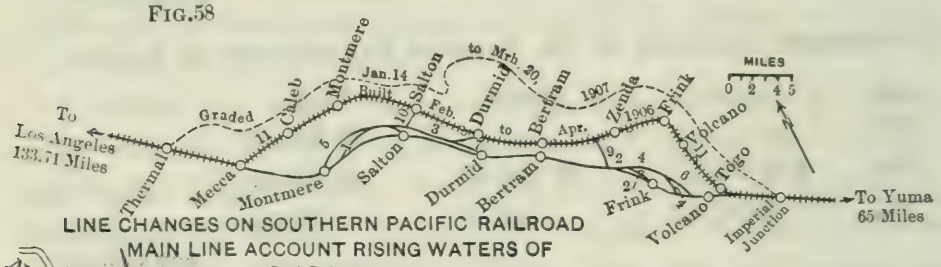


LOCATION OF THE SECOND BREAK DEC. 7, 1906 AND OF SECOND CLOSING JAN. AND FEB. 1907

FIG. 58

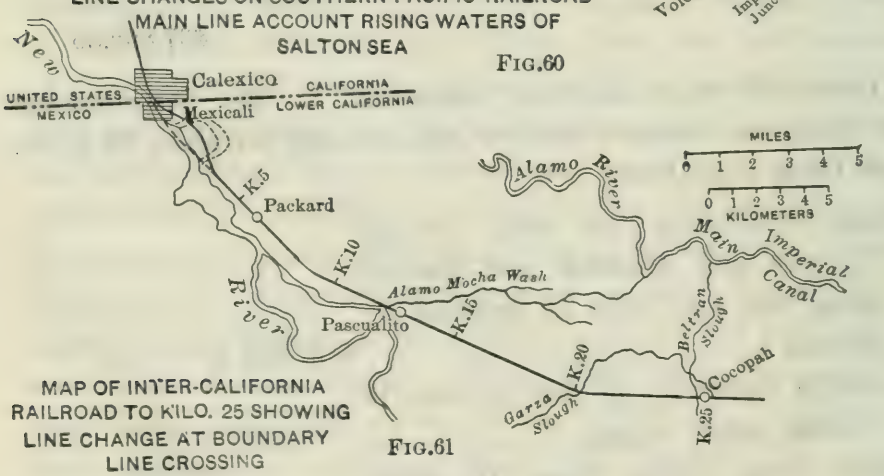
FIVE SHOOFLYS ON THE IMPERIAL BRANCH S.P. CO. AT CROSSING OF ALAMO RIVER AND THREE SHOOFLYS BESIDE THE NEW RIVER BETWEEN ROCKWOOD AND BRAWLEY

FIG. 59



LINE CHANGES ON SOUTHERN PACIFIC RAILROAD MAIN LINE ACCOUNT RISING WATERS OF SALTON SEA

FIG. 60



MAP OF INTER-CALIFORNIA RAILROAD TO KILO. 25 SHOWING LINE CHANGE AT BOUNDARY LINE CROSSING

FIG. 61

Thus the damage sustained by the Southern Pacific on permanent way alone, and not including interruptions to traffic, or expenses of any kind incurred by delayed trains, etc., was $762 664.50, and by the Inter-California $108 245.98, making a total of $870 910.48.

In addition to such damages, the trains of rock hauled during the 3 weeks of the first closing and more than 2 weeks of the second closing, or more than 5 weeks in all, were given rights of way over all except passenger trains, and the slower of these were very frequently delayed in order to hurry material to the front. The demand on the equipment of the road was tremendous, particularly during the second closing, when there were in rock-train service, 1 000 flatcars, 300 "battleships," 4 steam shovels, 10 work trains, exclusive of rock trains from the quarries other than at Andrade, etc. Indeed, for about 10 days, practically no freight was hauled out of the Port of San Pedro because of lack of equipment. The degree of the strain is shown by a telegram the writer received from the Superintendent of the Los Angeles Division, just before the second closing was accomplished, asking information as far in advance as possible "when work will slow up because I want to make arrangements to resume operating the division."

About 3 000 acres of cultivated land and 10 000 acres of uncultivated and public lands were practically destroyed and rendered unavailable for agricultural purposes under existing conditions, the total value of which depends on whether the present or prospective worth is considered. Perhaps a very fair figure would be $50 per acre for cultivated land and $10 per acre for raw lands, making the total damage to the land about $250 000.

Various individual settlers in Mexico and the United States suffered more or less severe injury from inundation of crops, etc., as distinct from land damages, and these probably amounted to not more than $150 000. A number of claims for damages sustained on the American side of the line were combined in a suit totaling $490 000, but this included damaged lands estimated at prospective rather than real and present figures. Judgment has just (October, 1912) been rendered for a total of $78 000.

The most serious injury done to settlers was the entire stoppage of the water supply in the canals of Districts Nos. 6 and 8 from the summer of 1906 until January, 1907. No claim was ever presented for these damages, and nothing more than crude guesses can be made as to

the amount. The result practically forced the depopulation of more than 30 000 acres of land, of which about 12 000 acres were under cultivation. The effect on the development of the valley at the time was not very great, but fear of repetition of a break, as much as anything else, has, until recently, retarded the region to a considerable extent.

Because of the tremendous expenditure involved in re-diverting and holding the river in its course, up to December 5th, 1907, when the Southern Pacific advanced funds for the work directly instead of to the C. D. Co., the original $200 000 loan to the C. D. Co. had grown to $1 100 000, and this was swelled by later bills, interest, etc., to approximately $1 375 000, by January 1st, 1909. In addition to owing this large sum for cash actually advanced, and for which payment could not be disputed successfully, a judgment for $458 246.23, in favor of the New Liverpool Salt Company had been rendered in the United States District Court toward the end of November, 1907, and there were claims from the Southern Pacific Company and others aggregating $1 360 000, two-thirds of which were from the latter company. The runaway river rendered hopelessly bankrupt the C. D. Co. and the Mexican Co.

Benefits.

To almost every cloud there is a silver lining, and this is no exception. It is now known that the diversion would have occurred very soon. The event showed the existence and nature of the danger and the necessity for guarding against it. Much more important was the development and standardizing of methods of closing future crevasses which might occur. Incidentally, the information of this character afforded to the Engineering Profession in general will doubtless prove of much value, though this cannot be considered as a benefit to the region in question.

By far the greatest benefit was the erosion of the great Alamo and New River barrancas and the creation of the main features of a complete and comprehensive drainage system for the entire Imperial Valley. The natural slope of the ground is remarkably uniform, with a grade of about 5 ft. per mile, and the very small, shallow channels of the New and Alamo Rivers were the only rudiments of satisfactory drainage, from an irrigation point of view. The Salton Sink is the natural drainage sump for the region, and its absolute control should have been acquired in the very beginning, either by the irrigating

company or by the land holders of the valley. In the litigation which followed the destruction of the salt works, the New Liverpool Salt Company, as owner of the submerged land, obtained a decree perpetually enjoining and restraining the C. D. Co. from diverting water from the Colorado River in excess of the substantial needs of the people dependent on the canal for water supply for domestic, irrigation, and such other lawful purposes as the same may be applied to, and with a further provision as to the control of the water diverted so that it will not overflow on the lands of the complainant. Later proceedings resulted in a most remarkable construction of the last portion of the injunction, so that now the Salt Company practically cannot object to the use of the basin as a natural sump.

TABLE 19.—Analysis of Water of New River.

Sample taken at Brawley, June 6th, 1908, and submitted by Mr. F. W. Roeding, Irrigation Investigations, Berkeley, Cal.

		Grains per gallon.	Parts per 1 000 000.
Alkali	Potassium Sulphate very small, and Sodium Sulphate (Glauber's salts), etc.	12.87	221
	Sodium Chloride (common salt)	389.40	6 675
	Sodium Carbonate (sal soda)	0.99	17
	Calcium Chloride	14.91	256
	Magnesium Chloride	37.26	639
	Calcium and Magnesium Carbonates, etc., large Calcium Sulphate (gypsum) chiefly	94.29	1 617
	Silica	1.22	21
	Organic matter chars, and chemically combined water	57.44	985
	Total	608.38	10 431

Providing for the region an efficient drainage system to carry all the waters into the Salton Sink would have required a large amount of money—so large that the date of its establishment would have been delayed very far into the future, much too far for the valley's real interests. This is true because it is plainly not the business of an irrigation company to supply a drainage system, and all other interests of the valley are very much divided because of the mutual water company plan of organization, and because of the usual lack of co-operation among farmers. Furthermore, the need for drainage of irrigated land is usually not recognized in time, and not admitted when it is recognized.

Indeed, in spite of the rather alkaline character of the lands in Imperial Valley, as already explained, it was not until November 1st, 1911, that any serious suggestion was made for a community drainage canal—in Imperial Water Company No. 8—the reasons then chiefly urged being:

"The loss of ground and bad appearances caused by the ends of the irrigated lands being covered with weeds or wild grass or perhaps nothing at all as the result of standing water."

A few spots in Imperial Valley are beginning to indicate an undesirable increase in alkalinity, and it is most fortunate that the magnificent main drainage ways of the Alamo and New River channels exist.

Salton Sea.

From a geological and spectacular point of view, the creation of the Salton Sea in so short a time was one of the most striking effects of the river diversion. The water filled the basin to a maximum elevation of —197.4 U. S. G. S. datum, or —204.2 S. P. datum, the maximum depth of water being 76 ft. The total area covered at this time was about 445 sq. miles, with a length of 50 miles and a width of from 10 to 15 miles. With the exception of the Great Salt Lake and Lake Michigan, the sea was the largest body of water lying wholly within the United States.

The water rose at the maximum rate during the latter part of June, 1906, when it gained nearly 7 in. per day, or 15.4 ft. during that month. From the reconnaissance map of the Salton Sink, published by the U. S. Geological Survey in 1905, it would appear that the areas of the various contours of the basin are about as follows:

Elevations.	Square Miles.
—280	0
—250	240
—200	445
—150	650
—100	920
— 50	1 150
— 0	1 750
+ 30 (old beach line)	1 950

TABLE 20.—Analyses of Water from the Salton Sea and the Colorado River at Various Seasons.

As made by Dr. W. H. Ross, of the Agricultural Experiment Station, University of Arizona, Tucson, Ariz.

	Salton water, surface of deepest portion, June 3d, 1907.	Colorado River water, low winter stage, January 10th–March 26th, 1900.	Colorado River water, rising summer stage, from melting snow, March 27th–April 30th, 1900.	Colorado River water, summer flood from melting snow, May 1st–June 29th, 1900.	Colorado River water, low summer stage, June 30th–August 26th, 1900.	Colorado River water, affected by local floods, August 27th–October 1st, 1900.	Colorado River water, summer floods from Arizona, October 2d–November 19th, 1900.	Colorado River water, low winter stage, November 20th–January 24th, 1901.
Total soluble solids at 110° Cent. Parts in 100 000 ..	364.8	92.9	67.4	32.2	36.1	71.4	104.5	87.1

Complete Analysis of Soluble Salts, Stated by Ions.
Parts in 100 000.

Sodium, Na	111.1	19.0	15.3	5.5	7.6	14.6	18.2	16.0
Potassium, K	2.3	1.1	2.1	1.0	1.3	1.8	2.1	1.2
Calcium, Ca	9.9	7.5	4.9	4.2	4.6	7.7	12.5	9.2
Magnesium, Mg	6.4	3.1	2.0	1.2	1.2	2.2	2.8	2.8
Aluminum, Al	0.031	....	....	....	....	....	....	....
Iron, Fe	0.005	....	....	....	....	....	....	....
Chlorine, Cl	169.7	20.5	13.9	4.5	6.9	15.8	17.5	18.1
Sulphuric, SO_4	47.6	26.1	19.4	7.2	7.6	19.7	35.6	23.8
Carbonic, CO_3	6.6	7.4	8.4	7.2	7.7	10.0	12.2	10.7
Silicic, SiO_3	1.2	4.6	2.1	2.1	3.3	2.3	2.2	2.1
Phosphoric, PO_4	0.018	....	....	....	....	....	....	....

Practically all the water which enters Salton Sea comes from the Alamo and New Rivers, which, under normal conditions, are now important chiefly as drainage channels for the Imperial Valley. Frequently, however, very heavy precipitation occurs in violent storms over small portions of the area draining into the basin, but the run-off, though occasionally of considerable quantity, is not relatively important. The total annual inflow is at present probably 200 000 acre-ft., or sufficient to cover the surface of the sea about 0.7 ft. in depth, while the evaporation is probably between 5½ and 6 ft. and the percolation insignificant.

TABLE 21.—Composition of Ocean Water.

(This table gives the mean of 77 analyses made by the Challenger Expedition, Challenger Report, Physics and Chemistry, Vol. 1, 1884, p. 203.)

Stated by Ions. Parts per 100 000.

Sodium (Na)	1 071
Potassium (K)	39
Calcium (Ca)	42
Magnesium (Mg)	130
Sulphate (SO_4)	270
Chloride (Cl)	1 935
Bromide (Br)	6
Carbonate (CO_3)	7
	3 500

Quite a little speculation has been indulged in regarding the length of time which would have been required to fill the Salton Sea had the Colorado River not been re-diverted. Most of such computations are based on too low an average flow of the river past Yuma, which it now

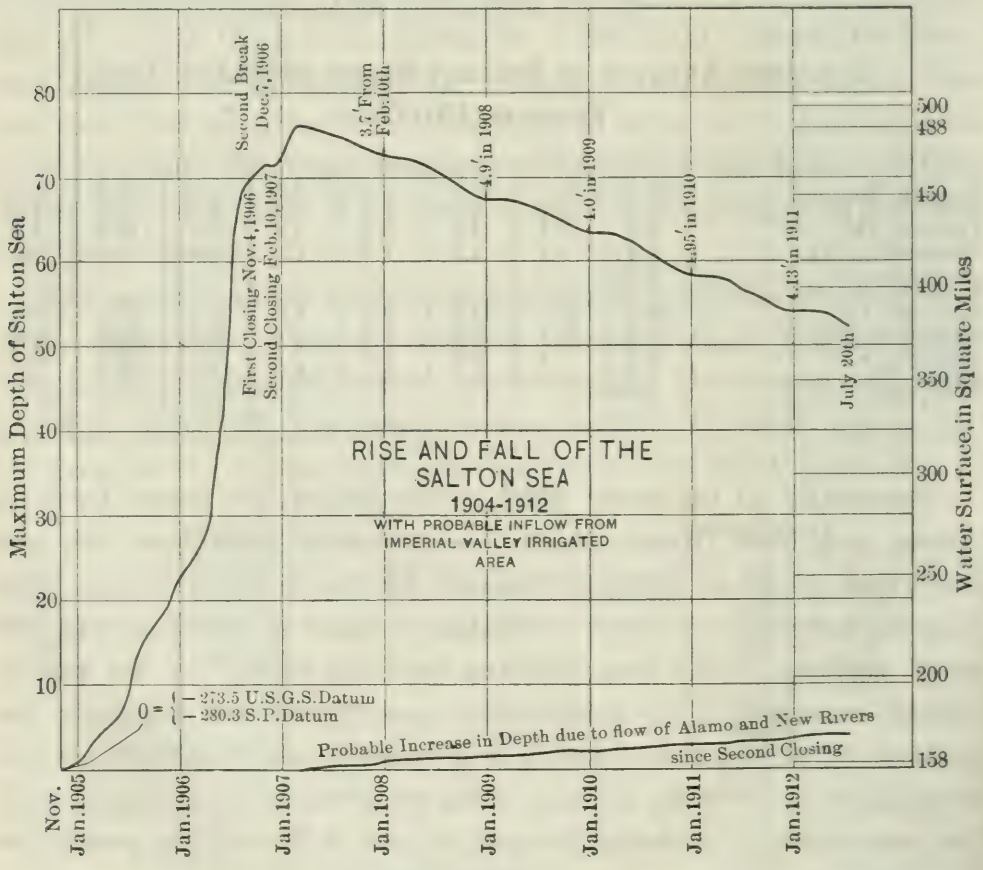


Fig. 62.

seems is in excess of 12 000 000 acre-ft. per annum. As a matter of fact, however, the inflow from the Imperial Valley region will constantly increase, and the quantity evaporated will decrease directly with the decrease in water surface exposed, so that a balance will be reached probably in such time as the inflow will average between 350 000 and 500 000 acre-ft. per annum from all sources, and the exposed surface will cover between 60 000 and 80 000 acres. At such time the maximum depth of the sea will be between 8 and 10 ft.

The sea has already (January, 1912) fallen about 22 ft., and has exposed approximately 115 sq. miles which were under water. The salt beds were dissolved to such an extent as to render the water of the sea quite salt, unfit entirely for drinking purposes, and it was assumed that the land which it covered would be hopelessly alkaline. This does not seem to be the case, and a very considerable acreage of such exposed land is being cultivated with entire success.

Much speculation was indulged in regarding the effect of this body of water on the rainfall and climate of the Southwest. A careless consideration of the precipitation on the drainage area of the river, particularly that of the Gila water-shed, before and after January, 1905, might lead to the conclusion that the effect is quite marked. The period from January 1st, 1905, to date has been one of very heavy rainfall throughout the Southwest, its most remarkable part being early in January, 1905, which was before the formation of the Salton Sea. Professor Alfred J. Henry* points out the fallacy of such an opinion, as follows:

"Admitting, for the sake of argument, that a body of water * * * existed * * * 60 miles long, 8 miles broad, and say 25 feet deep on the average. * * * The cubic contents would therefore be $60 \times 8 \times 0.0047 = 2.2$ cubic miles of water. The normal annual rainfall of Arizona * * * is 11.75 inches [while in 1905 it was 26.6 inches], or an excess of 14.85 inches, an amount more than equal to the normal annual rainfall. * * * As the area of the Territory is 113,956 square miles, * * * the number of cubic miles of rain that fell in Arizona in excess of the average was * * * 27, * * * twelve times greater than the total volume of the Salton Sea. In other words, the total volume of the latter would barely suffice to produce one-twelfth of the surplus rain that fell in Arizona, to say nothing of the rainfall in adjoining regions. The total

* "The Salton Sea and the Rainfall of the Southwest," *Monthly Weather Review*, Washington, December, 1906.

amount of water now in Salton Sea, if uniformly distributed in Arizona, would cover the Territory to the depth of about an inch and a quarter, or the equivalent of one good soaking rain."

As a matter of fact, the area of the Salton Sea and Laguna Maquata combined are insignificant when compared with that of the Gulf of California, and are just about as far from Arizona. Professor Henry concludes that the Salton Sea has increased the relative humidity in the immediate vicinity in a slight measure; that it is improbable that any considerable portion of the vapor it gives off passes beyond the immediate confines of the desert; and that there might be a tendency toward lower maximum and higher minimum temperatures in a narrow zone immediately surrounding the sea, particularly on the leeward side.

International Negotiations.

When the Mexican Co. obtained its concession from the Mexican Government, Col. Jacobo Blanco, then Chief of the International Boundary Line Commission for the Mexican Government, with headquarters at El Paso, was appointed Inspector of the Mexican Co. and its operations. In 1906, Col. Blanco died, and his successor on the International Boundary Line Commission was Señor Fernando Beltran y Puga, who was also appointed his successor as Inspector of the Mexican Co. The writer considers this appointment an exceedingly fortunate one, as Señor Beltran y Puga is an exceptionally efficient, aggressive, and fair-minded man, and an engineer, with whom it has always been a satisfaction to transact business. Immediately on his appointment he acquainted himself with the conditions along the river and with the affairs of the Mexican Co., and has always acted promptly and with decision.

In the spring of 1908 the United States State Department appointed Mr. Louis C. Hill, Supervising Engineer of the U. S. Reclamation Service, to represent the United States on a joint commission to work out the provisions of a treaty with Mexico for the control of the Lower Colorado River and an equitable distribution of its waters. This appointment was in line with President Roosevelt's promise to Mr. Harriman, and was doubtless in a measure brought about at this particular time by the failure of Congress to take any action on the bill to reimburse the Harriman interests. At the re-

quest of the United States, the State Department of Mexico appointed a Commissioner, and rather naturally selected Señor Beltran y Puga to represent the Mexican Government, this gentleman's appointment being made on May 7th, 1908, and practically simultaneous with the appointment of Mr. Hill. Both gentlemen were instructed to act together and make a study of the works and operations necessary to complete international control of the lower Colorado River and render impossible a repetition of the recent disaster and the complete utilization of the waters of the river, such study to be in whatever detail might be deemed necessary.

This Commission never had a formal meeting, which is very much to be regretted, considering the importance of the matter. Very shortly after their appointment, the Commissioners had an informal meeting at which, according to private conversations which the writer had with both gentlemen, it appears that Señor Puga submitted, in the form of a written memorandum as the basis for discussion, the suggestion that both Governments cancel the existing treaties regarding the navigability of the river; that regulation of the flow of the river by extensive storage works in the upper portions of the drainage basin was desirable; that both Governments determine the priority and extent of existing water rights and fix rules for granting future water rights; that a joint international commission should make all engineering and other investigations necessary, and divide the costs thereof; that all plans or projects existing or proposed along the river should be submitted to the investigation of the joint commission; that a report be made outlining, in a general way, the work to be done, for the purpose of having a full and complete treaty arranged and the necessary definite appropriations set forth; and that it would be agreeable for Mexico to negotiate a treaty, either preliminary or final, at the earliest possible moment. It seems, however, that Mr. Hill could not find time to attend to the matter, nor were special funds placed at his disposal to defray the necessary expenses. At any rate, two years later, nothing having been done, on May 17th, 1910, the United States Department of State replaced Mr. Hill with Mr. Wilbur Keblinger, Secretary of the American side of the International Boundary Commission, with offices in El Paso and Washington. This change of American Commissioners, however, has not had any apparent result thus far.

It is hoped that discussion of this paper may bring out the explanation for the failure of these representatives of the two governments even to have a formal meeting in the 4½ years which have elapsed since the Joint Commission was appointed at the request of the State Department of the United States. Such explanation would be particularly interesting, because the writer knows Mr. Hill to be an unusually tactful, courteous, and aggressive gentleman, and an efficient engineer, and understands that Mr. Keblinger is also an unusually capable man.

In any event, there seems to be no doubt that the Mexican Government has been, and is, not only willing, but anxious, to arrange for a satisfactory joint control of the Lower Colorado River, and that the responsibility for nothing definite having been accomplished in this direction, in large measure at least, rests with the United States.

Building of Volcano Lake Levee.

The extraordinary quantity of water which got into Volcano Lake during the summer flood of 1907 raised it higher than it had ever before been known to be, and a large quantity of water passed northward through the New River outlet. Furthermore, a reconnaissance showed that the large quantity of overflow water had started cutting back fingers from the Volcano Lake region toward the river, which indicated the probability of the diversion of the Colorado River below the divide of the delta cone, along the Pescadero, Abejas, or Paredones. Therefore it seemed that another portion of the complete levee system, as originally planned by the writer to hold back the overflow waters of the Colorado from the Salton Basin, should be constructed—that portion from the mountains on the west side of the valley eastward along the north of Volcano Lake to the low-lying divide or ridge farther on. The C. M. Co., however, objected to this, as Volcano Lake is entirely on its land and its utilization for irrigating a portion of that company's lands was considered, but found impracticable because of the great variation in the water surface, the inundation of a part of the land, however, at flood times was simple, and permitted securing considerable pasturage. The C. M. Co., however, was willing to give the right of way and permit the construction of the protective works provided a permanent head-gate were installed at Cerro Prieto through which water might be let into the New River when the lake was

PLATE LI.
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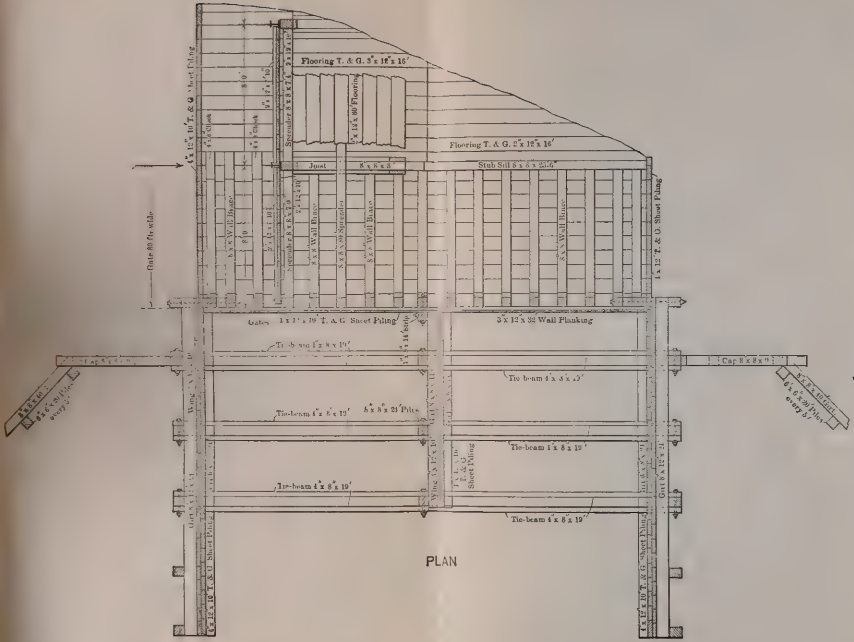
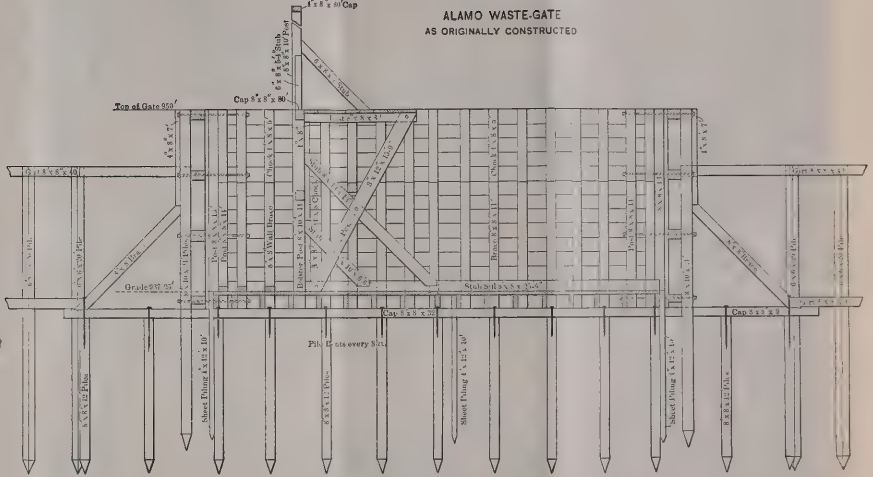
1418A.

ALAMO WASTE-GATE
AS ORIGINALLY CONSTRUCTED



PLAN



ELEVATION

full. This was finally agreed to, and arrangements were completed under which the Southern Pacific Company agreed to pay the Mexican Co. for constructing this gate on condition that the Mexican Co. would arrange to have 8 miles of levee to the east built. This was done and the gate and levee were completed just before the summer flood of 1908 began to throw its waters into Volcano Lake. After one season the Mexican Government compelled the removal of this intake gate and the levee to be built around in front of it, so that it is not now in service. Plans for the gate and levees were presented informally to the Mexican Government Inspector of the Mexican Co., Señor Puga, and it was understood that their construction was approved and permission verbally given to begin work before filing maps and drawings and having them approved or changed as required by the Departmento de Fomento in such cases—this on account of emergency. It was not so understood by Señor Puga, and the construction of the gate was a needless expense.

All this work was arranged for and practically done while Congress was considering the payment of the bills for the second closing and subsequent protective work, and when there was no reason to doubt that a fair adjustment would be made.

Reconstruction of Irrigation System.

As soon as the river control work was assured, arrangements were made to ascertain the exact condition of the irrigation system of the valley, and what was necessary and advisable to do in connection with it. Accordingly, Mr. F. C. Herrmann, who was added to the engineering staff on February 1st, 1907, was placed in charge of this work. The damage done by the flood was confined almost entirely to carrying away the flume by which the West Side Main Canal crossed New River, and a similar flume, 20 miles north over New River, carrying water from the Central Main to supply Imperial Water Company No. 8. To rebuild the latter was impracticable on account of the immense barranca which had been created at the old crossing, and it was decided to enlarge the West Side Main and extend it northward so that all the territory west of New River would be served thereby.

A wooden flume, supported by wooden piling, was designed to cross the New River gorge very close to the location of the original flume,

carried away about March, 1906. Work was begun on the structure and rushed to completion. This flume is worthy of note because of its height, length, and cost, as a quasi-temporary structure. It is 1 860 ft. long and the maximum height of the trestle is 55 ft. It supports a rectangular flume, 16 ft. wide and 6 ft. deep, built of 2-in. redwood lumber with ship-lap joints. It has given excellent service, and the leakage has been notably slight from the time it was first put into service.

Surveys for the reconstruction, enlargement, and extension of the West Side Main were hurried, and contracts were let for the work, which was well under way when the financial panic of November, 1907, occurred. The contractors were forced into bankruptcy, and the work was completed by their bankruptcy trustee, which caused considerable delay, but water was turned through the reconstructed West Side Main late in December, 1907. This canal is 28 miles long—7 miles in Mexico and 21 in California—and has a capacity varying from 800 to 400 sec-ft., with 760 000 cu. yd. of earthwork moved at a total cost of $86 000, and $5 000 for two temporary structures.

Some little time later the Rose Levee, across the Alamo channel at Holtville, was reconstructed, with a waste-gate to pass the excess of water coming through the Holton Power Company's plant and through the Alamo Waste-gate farther up that channel in Mexico, and a head-gate for the Rositas Canal. Both of these are of reinforced concrete, the waste-gate being of interesting design and capable of passing 2 000 sec-ft. with a total drop of 20 ft. In this way water which must be furnished to the Holton Power Company under its contract is picked up below the plant and utilized for irrigation, as was the original intention when the contract was made. These two permanent structures and the earthen dam cost $55 250.

Another important permanent structure, known as the Seven-Foot Drop (Plates LII, LIII, and Fig. 63), was built in the Central Main just south of the Boundary Line at a cost of $23 760, including three small structures adjacent to it, but not exactly a part thereof. This structure takes the place of a wooden 10-ft. drop nearly 2 miles farther on, which, by the way, had passed considerably more water than it was designed for and was in fairly good condition when removed after 8 years of service.

The chute of the Alamo Waste-gate was repaired and again extended down stream, the result being a quite remarkable construction, Plate

1420A.

PLATE LII.
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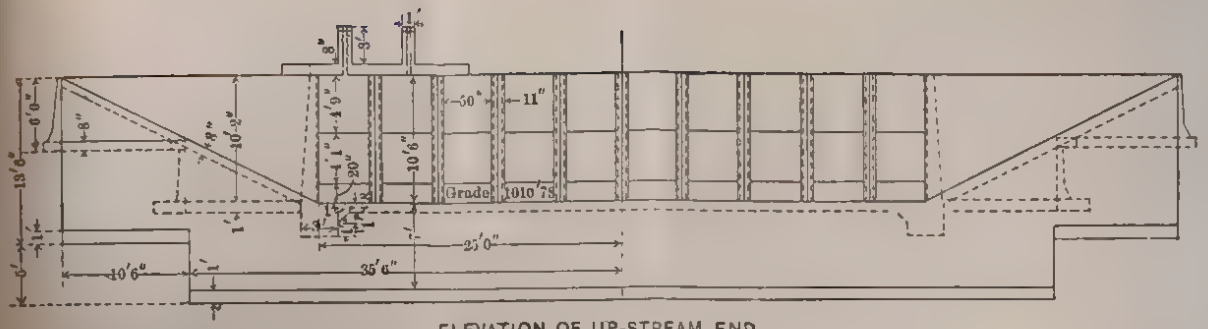
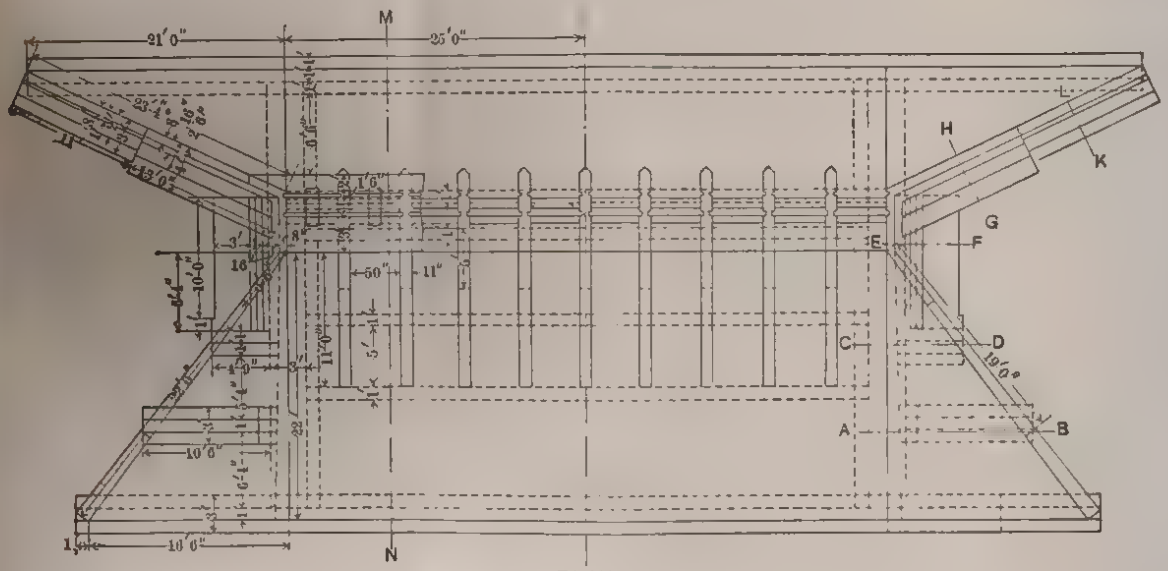
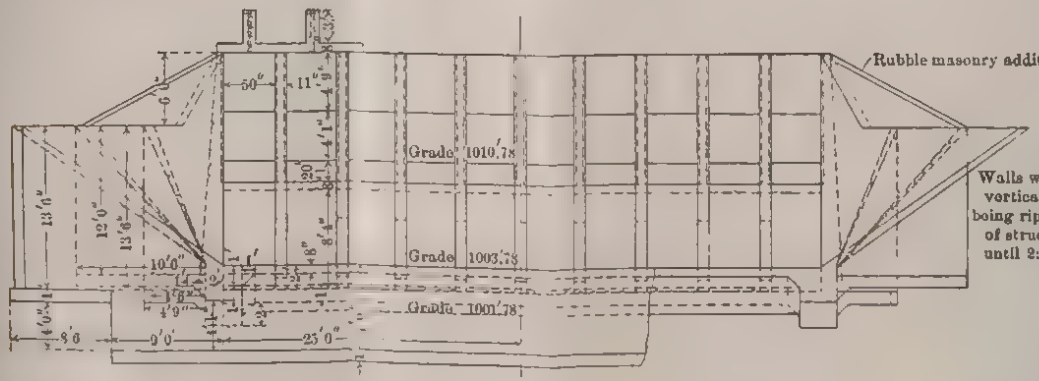


ELEVATION OF UP-STREAM END



PLAN



Walls warped from vertical to 2:1 slope; being rip-rapped from end of structure as shown, until 2:1 slope is reached.

ELEVATION OF DOWN-STREAM END

LI, which has never shown any signs of weakness against a head of approximately 35 ft.

Surveys and Designs for Extensions.—At the same time, surveying parties were assembled and topography was taken, with 1-ft. contour intervals, on 230 sq. miles. This included the west side of New River and a strip in Mexico adjoining the Boundary Line averaging 3 miles wide and running from the West Side Main crossing of the Boundary Line to about 4 miles east of Sharp's Heading and thence generally following the Alamo channel almost to the levee system along the river, the Alamo channel being carefully mapped and cross-sectioned. Much of this work was done in the heat of the summer, five complete surveying parties being on the work until after August 1st, when the force gradually lessened. A large part of the area was practically cleared land, but much of it was covered with a dense undergrowth, which made progress very slow, and, by cutting off the breeze, accentuated the severe climatic conditions. Nevertheless, the cost of this work was about 57½ cents per acre, or $37 per sq. mile. This experience showed that, while the great summer heat there is quite disagreeable, it does not render engineering field work by any means impracticable, and indeed does not increase the cost more than 10 per cent. One agreeable feature of the very hot season is that the temperature is too great for flies, so that they practically disappear.

Cross-sections and current-meter observations were taken at various points in the Main Canal and in the important canals of the distribution systems of the various mutual water companies, to determine capacities, losses, etc.

When these data were compiled and put into form, estimates were made for the reconstruction and enlargement of the existing system and for extensions to cover a great deal of new territory. The estimate for this work was approximately $900 000 with temporary structures, and including considerable improvements in the main Alamo channel from the river to the controlling works in the valley; and $2 200 000 with permanent structures at essential points and replacing the Alamo channel with an entirely new twin main canal from a few miles below the concrete head-gate to the mesa ground east of Sharp's Heading. Whatever possibility there was of such fundamental reconstruction and extensions was dissipated by the severe financial stringency in November, 1907.

These designs included the construction of a head-gate on Mexican territory, and the total abandonment of the concrete head-gate, along with diversion, on American soil. The head-gate thus proposed was just behind the levee, about 3 miles below the Boundary Line, with a caisson foundation supporting extensive wing-walls and massive piers, between which there were to be large vertical gates, operated by gasoline motors.

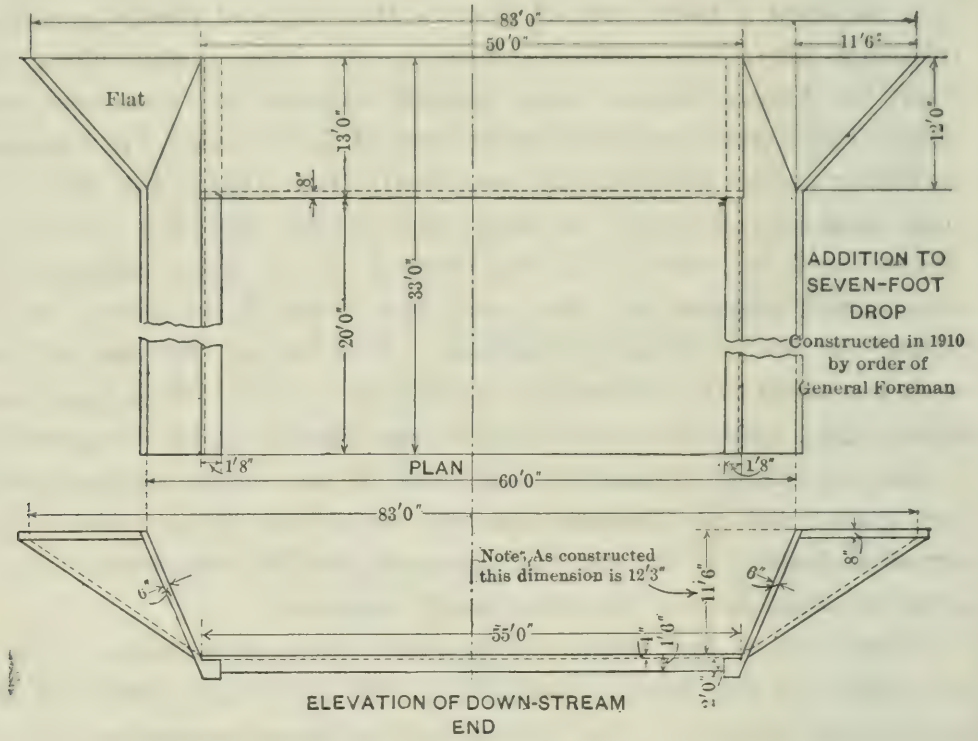


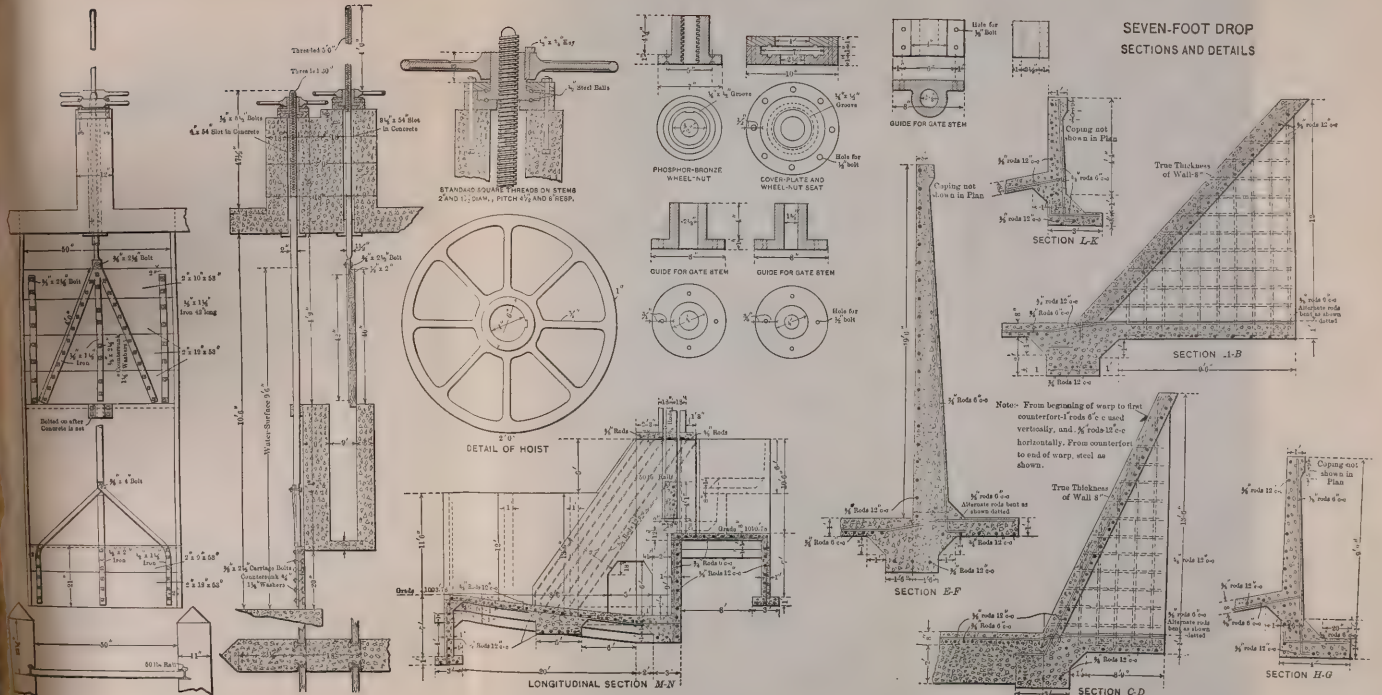
FIG. 63.

Of these designs the only ones constructed were the West Side Main, the Rose Levee and Waste-gate, including the Rositas Heading on the Alamo near Holtville, and the Seven-Foot Drop in the Central Main. The foundations in all cases were merely concrete footings without any sheet-piling, of which the Seven-Foot Drop (Plates LII and LIII, and Fig. 63), the Rositas Waste-gate (Fig. 64 and Plate LIV), and the Rositas Head-gate (Figs. 65 and 66) are quite typical.

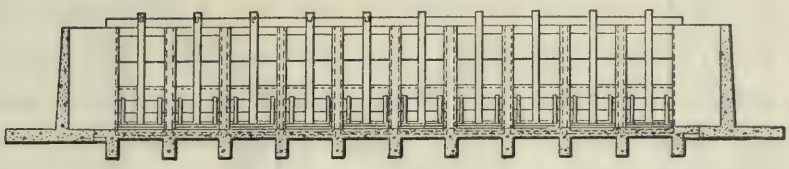
The Rositas Heading and Waste-gate together cost $55 250, and the Seven-Foot Drop, together with the little gates which constitute one structure, cost $23 750. Concrete work, generally, has cost about three times as much as wooden structures; it has ranged from $30 to $35 per cu. yd. for the entire cost of completed structures.

1422A.

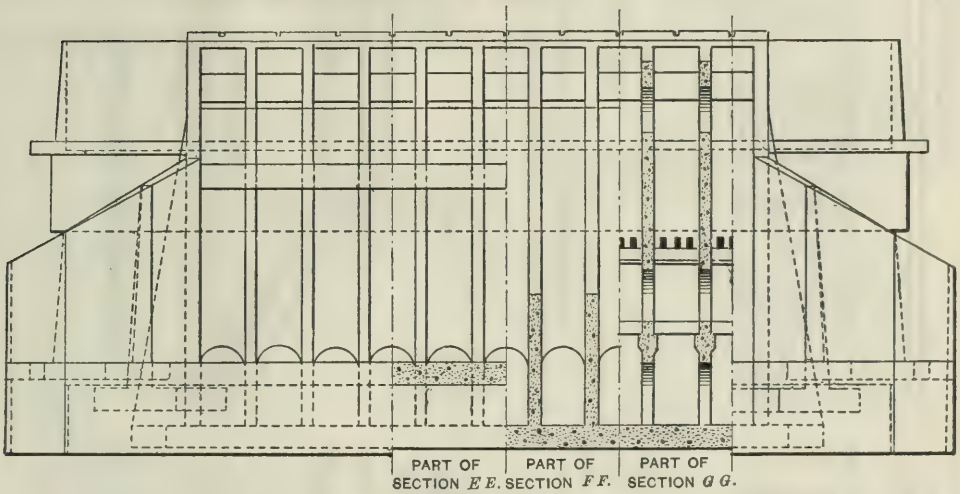
SEVEN-FOOT DROP
SECTIONS AND DETAILS


Threaded 3'0"
Thread 1-1.20"
Holes 5/8" x 1 1/4"
4 1/2 x 54" Slot in Concrete
9 1/4 x 54" Slot in Concrete
Bolted on after Concrete is set
Water Surface 5'6"
50 lb. Rail
STANDARD SQUARE THREADS ON STEMS 2" AND 1 1/2" DIAM., 1 PITCH 4 1/2" AND 6" RESP.
1/2" Steel Balls
PHOSPHOR-BRONZE WHEEL-NUT
COVER-PLATE AND WHEEL-NUT SEAT
GUIDE FOR GATE STEM
GUIDE FOR GATE STEM
Hole for 1/8" bolt
GUIDE FOR GATE STEM
DETAIL OF HOIST
LONGITUDINAL SECTION M-N
Coping not shown in Plan
SECTION L-K
True Thickness of Wall-8"
SECTION A-B
Note:- From beginning of warp to first counterfort rods 6'0" c/c used vertically, and 3/4" rods 12'0" c/c horizontally. From counterfort to end of warp, steel as shown.
SECTION E-F
SECTION C-D
SECTION H-G

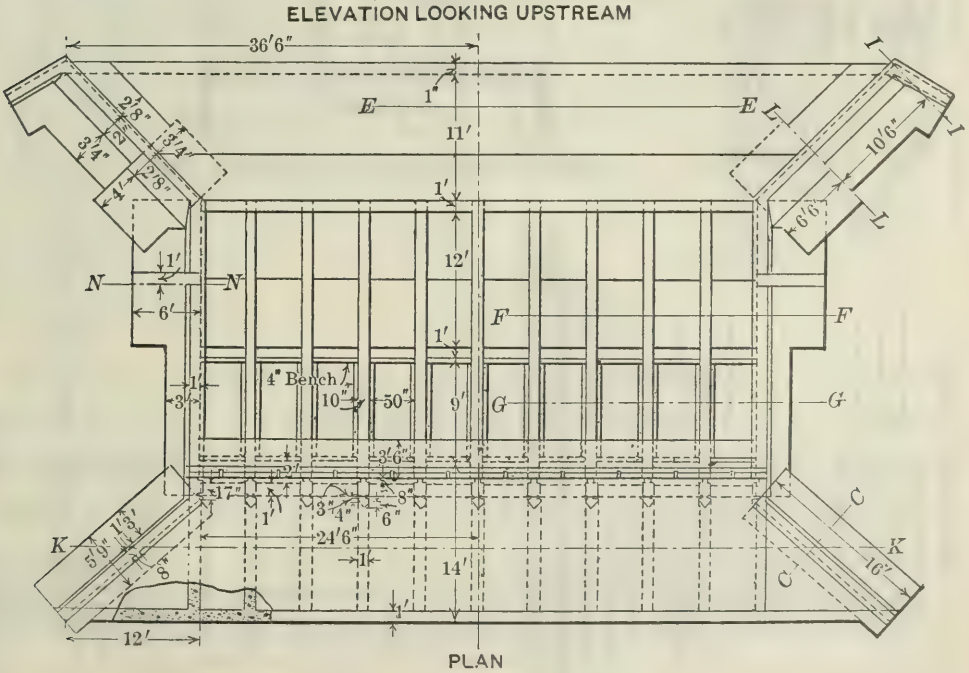
ROSITAS WASTE-GATE



SECTION *K K* AND ELEVATION OF GATES



ELEVATION LOOKING UPSTREAM



PLAN

FIG. 64.

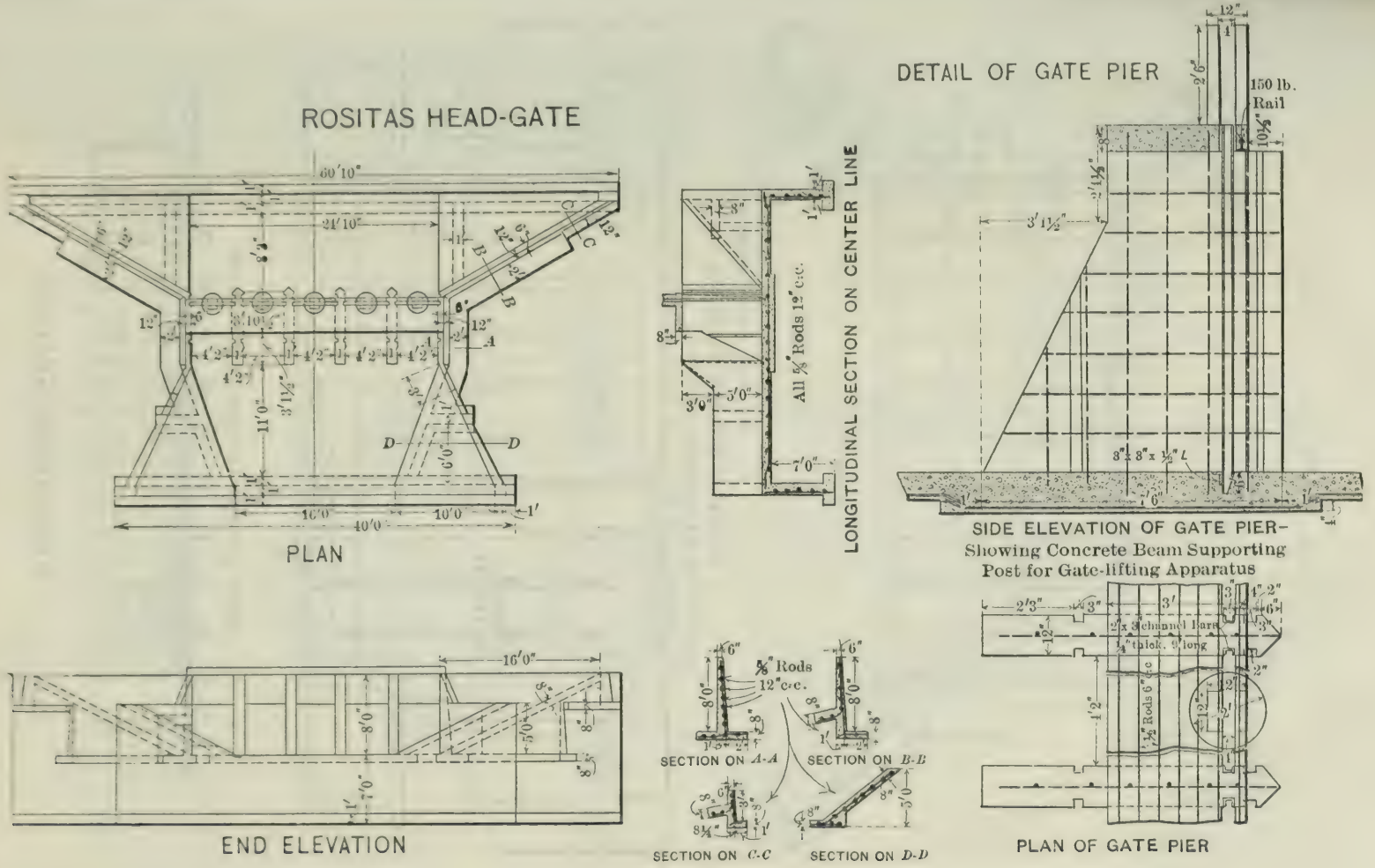

ROSITAS HEAD-GATE
PLAN
END ELEVATION
LONGITUDINAL SECTION ON CENTER LINE
All ⅝" Rods 12" c.c.
DETAIL OF GATE PIER
150 lb. Rail
SIDE ELEVATION OF GATE PIER—
Showing Concrete Beam Supporting
Post for Gate-lifting Apparatus
PLAN OF GATE PIER
⅝" Rods
12" c.c.
SECTION ON A-A
SECTION ON B-B
SECTION ON C-C
SECTION ON D-D

FIG. 65.

Maintenance and Operation.

Until such time as the promised arrangements for river control and levee maintenance by the Government should be fulfilled, the operations of the irrigation interests, under instructions from higher authorities, were made to include levee maintenance and extension as well as supplying water in wholesale to the mutual water companies. The business done by the corporations was in this way much more varied than that of most irrigation companies, although none of the land they owned was under cultivation.

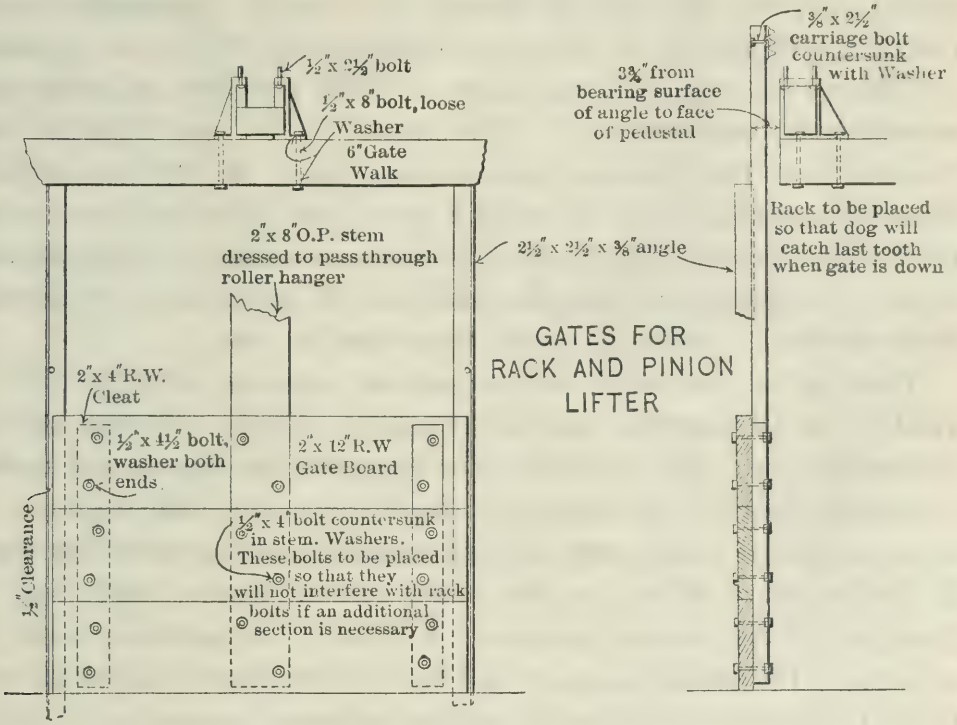

½"x 2½" bolt
½"x 8" bolt, loose Washer
6" Gate Walk
2"x 8" O.P. stem dressed to pass through roller hanger
2"x 4" R.W. Cleat
½"x 4½" bolt, washer both ends
2"x 12" R.W Gate Board
½" Clearance
½"x 4" bolt countersunk in stem. Washers. These bolts to be placed so that they will not interfere with rack bolts if an additional section is necessary
3⅜" from bearing surface of angle to face of pedestal
⅜"x 2½" carriage bolt countersunk with Washer
2½"x 2½"x ⅜" angle
Rack to be placed so that dog will catch last tooth when gate is down
GATES FOR RACK AND PINION LIFTER

Fig. 66.

Because of the litigation which seemed inevitable, and indeed which had started ere this, it was deemed essential to have a complete and satisfactory system of accounts. Inquiries were directed to all possible sources of information, including the U. S. Reclamation Service, to discover a system of irrigation accounts similar in a general way to the system of railroad accounts generally adopted and for some time past made obligatory by the Interstate Commerce Commission. No such system was found. The U. S. Reclamation Service has a fairly satisfactory system of construction accounts, but its work, as yet, is almost exclusively construction.

The accounting system, therefore, was worked out, modeled closely after the classified railroad accounts. The account numbers of the Mexican Co. are the same as those of the C. D. Co., except that one thousand is added. Four years' experience with it has resulted in few changes, and in its present form it is extremely satisfactory. Lack of space forbids giving it in full, as there are 146 expenditure and 16 revenue accounts.

In order to ascertain the cost of particular portions of the work—whether new construction, betterments, or ordinary maintenance—special accounts are kept as desired, by General Manager's Orders (G. M. O.'s). A G. M. O. is asked to secure authority for, or to secure cost figures on, any particular piece of work, and they are numbered consecutively, beginning with 1 for the C. D. Co. and 1 001 for the Mexican Co. The classified accounts and the G. M. O.'s are entirely independent, the latter being really a second and additional accounting for the same expenditure. G. M. O.'s were never asked for expenditures of less than $300, and were required for all expenditures of certain kinds specified in bulletins issued from time to time.

Distribution was made both to account numbers and to G. M. O. numbers on all payrolls, material requisition blanks, vouchers paying contractors, etc. The auditor's office gathers these figures and makes a monthly report, to the general manager and to the chief engineer, of the expenditures under each account number, totaling under headings. A similar report, of the receipts, under account letters, with totals, is furnished to the general manager. A monthly statement is made up and sent to the general manager and the chief engineer concerning each G. M. O., giving a statement of original estimate, amount expended to date, and percentage completed. Whenever work covered by G. M. O.'s is materially changed, a new G. M. O. is gotten out accordingly.

With such accounts, occasional trips of inspection, to observe the physical condition of the system, will afford a very complete knowledge of the exact condition of the project at all times.

The relationship between the C. D. Co. and the various mutual water companies was at all times very satisfactory and cordial, until the former was thrown into the hands of a receiver, there being one exception, however, in Imperial Water Company No. 5. Differences antedating the beginning of the railroad management and in a large measure due to dissensions between former President A. H. Heber and

PLATE LIV.
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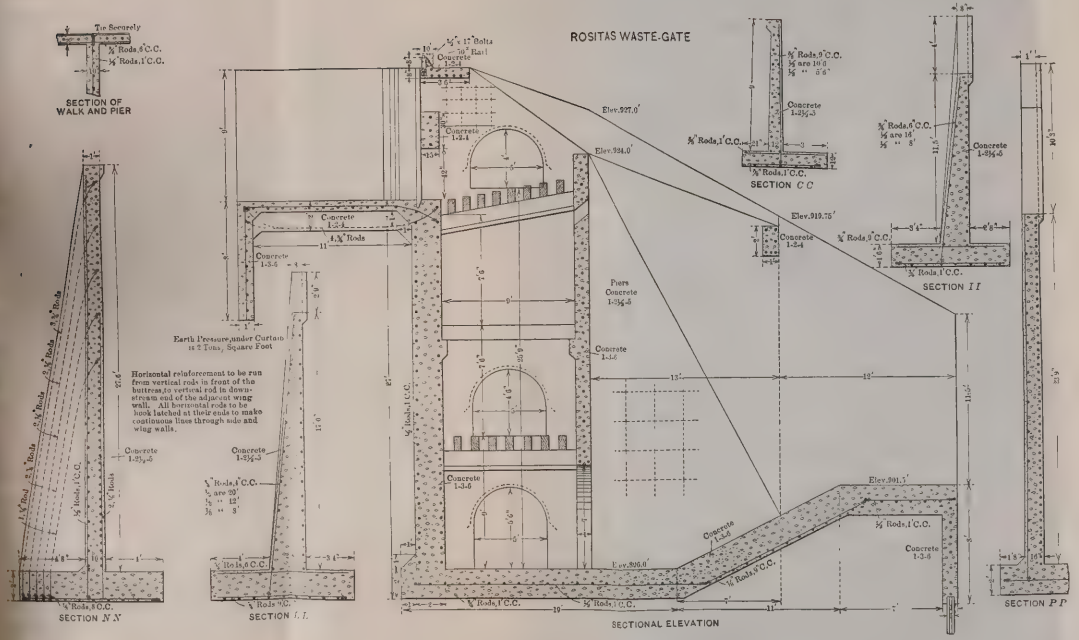
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.


ROSITAS WASTE-GATE
Tie Securely
⅜" Rods, 5' C.C.
½" Rods, 1' C.C.
SECTION OF WALK AND PIER
½" x 12" Bolts
50" Bars
Concrete 1-2-4
Elev. 927.0'
Elev. 924.0'
Elev. 919.75'
Elev. 901.7'
Elev. 896.0'
Concrete 1-2½-3
Concrete 1-3-5
Concrete 1-2-4
Piers Concrete 1-2½-3
Concrete 1-3-6
Concrete 1-2½-3
Concrete 1-3-5
4-¾" Rods
Earth Pressure under Curtain 2 Tons, Square Foot
Horizontal reinforcement to be run from vertical rods in front of the buttress to vertical rod in downstream end of the adjacent wing wall. All horizontal rods to be hook latched at their ends to make continuous lines through side and wing walls.
½" Rods, 1' C.C.
½" Rods, 8' C.C.
½" Rods, 6' C.C.
½" Rods, 9' C.C.
½ are 10'
½ " 5'6"
⅜" Rods, 5' C.C.
½ are 16'
½ " 8'
SECTION N N
SECTION I. I.
SECTION C C
SECTION II
SECTION P P
SECTIONAL ELEVATION

his Board of Directors, resulted in commencing litigation to compel Imperial Water Company No. 5 to pay up back water rentals. The attorneys of the latter company, in their cross complaint, attacked the validity of the mutual water company and water stock plan of a water right. The suit was before the United States Circuit Court, and in rendering the decision the judge expressed the opinion that the plan was illegal, and, practically speaking, the C. D. Co. was a public service corporation. This was but an opinion, because the State Courts of California have to decide this question finally, as it is a matter of the Constitution and Statutes of the State of California exclusively. The effect of this opinion, however, was to make the Southern Pacific Company feel that it would be unsafe to advance the large sums of money needed to reconstruct and extend the system on the expectation of being repaid ultimately through the sale of additional water stock to cover the new territory which would be brought under the canal. This suit, as much as any other factor, is responsible for the fact that practically nothing has been done on the betterment and extension work outlined. This litigation with No. 5, although begun in August, 1906, has not yet reached a definite conclusion.

When Imperial Water Company No. 1 was organized, the capital stock was placed at 100 000 shares, and the territory embraced within its limits was 135 000 acres. Of the excess land, a large part was found to be well worth reclamation, so that early in 1910 no No. 1 water stock was available. Imperial Water Company No. 1 was unwilling to increase the capital stock without obtaining from the C. D. Co. a contract increasing the quantity of water it would be entitled to demand and so retain the basis of 4 acre-ft. per annum per share of stock. It was impossible for the mutual water companies and the C. D. Co. to agree as to the proper division of the "water right" receipts for such 35 000 acres, so no relief was afforded to *bona fide* settlers who had their land ready and had to have water or lose their filings under the rulings of the U. S. Land Department. A trial suit was instituted, known as Thayer *vs.* C. D. Co., which was decided in favor of the plaintiff on March 17th, 1911, by Judge George H. Hutton, of Los Angeles, sitting for Judge Cole in the Superior Court of Imperial County. Judge Hutton in brief decided that the C. D. Co. was a public service corporation; that the rate for supplying water was 50 cents per acre-ft.; that the C. D. Co. was in a position

to supply the plaintiff without detriment to the other water users; and ordered that it do so. This decision has been appealed to the State Supreme Court. If it is upheld, the mutual water company will not be a necessary factor in obtaining water from the C. D. Co. This decision has not affected the price of water stock in the valley very materially, because the various mutual water companies own the distribution systems, and the difficulties of getting water from the short mileage of the C. D. Co. main canals through other sources than the distribution systems of the mutual water companies are practically prohibitive. The decision, of course, absolutely precludes extension of the irrigated area any farther through such a water stock plan.

Seepage and Irrigation Losses.—It is very unfortunate that a clause in all the triparty contracts requires the actual seepage and evaporation from each mutual water company's distribution system to be reported after a period of 3 years, and that the percentage allowance thus determined should thereafter be made, to the end that the mutual water companies would pay for the quantity of water which could actually be delivered to the individual settlers. All the mutual water companies joined and selected an engineer, Mr. F. C. Scobey, to represent them in making seepage and evaporation determinations, and did quite a little experimenting and investigating. The engineers of the C. D. Co. also made numerous determinations under the direction of Mr. Herrmann, but for various reasons no experimentation was done jointly. The amount of work and expense involved greatly surprised the representatives of the mutual water companies, and proposals of a compromise were made, one thing helping to this being that such representatives agreed that it would be distinctly unsatisfactory to have different percentages agreed on for each of the mutual companies, as must obviously be the result. The C. D. Co. was thrown into the hands of a receiver just as these negotiations were reaching conclusion on the basis of 6% flat allowance. In the confusion following, the companies insisted on a 10% allowance, and this is the present status.

The results obtained by Mr. Scobey have never been given out, but the experiments by the engineers of the C. D. Co. showed a surprisingly small loss. It appears that the very fine silt deposited in the distribution systems of the various mutual companies forms a practically impervious coating on the sides and bottom, the measured

ss in many cases being as small as 0.25% per mile and increasing a maximum of 1% per mile. From the available experimental data, mputation of the losses from the distribution system of Imperial Vater Company No. 1, comprising nearly 375 miles of canals, gives ie total for seepage and evaporation throughout the year, under pres-nt operating conditions, as somewhat less than 7 per cent. The writer as no knowledge of so small a loss by seepage being reported by any uthority heretofore.

The experience has been that the mutual water company plan of rganization to distribute water, obtaining it from a larger company t wholesale, is highly satisfactory, and it is commended for careful nsideration by those who are contemplating irrigation work, if the cal laws will permit.

Canal Maintenance.—As far as the physical maintenance of the anal system is concerned, it may be said that the average life of ie redwood structures, consisting of checks, drops, turn-outs, waste-ates, etc., is about 5 years for the smaller structures and 8 or 10 ears for the larger ones. It must be remembered that water is sed every day in the year, so that this experience has narrow appli-ation. The chief deterioration is caused by a sort of dry rot eginning about 1 ft. below the surface of the ground and ex-nding down not more than 2½ ft. It appears that deeper than iat, regardless of the quantity of water present, the heat does ot become great enough to cause trouble, and the upper layer f earth is nearly always so dry that the wood is not attacked. In ie thin intermediate layer, which is both damp and hot, and perhaps here enough oxygen is available, dry rot appears very rapidly, and ood wet on one side and in contact with earth on the other has the rth side damaged to a depth of from ¼ to ¾ in., sometimes in 9 or 12 onths. Redwood subjected to several kinds of treatment has been ied, but with little success. Oregon pine, for that portion of struc-ires covered by earth in that region, rots very rapidly indeed.

The chief lesson taught by the maintenance of the canals—other an that the cost is unusually large because of tule growths and silt—that inside berms must be avoided, and when possible all canals xcept the sub-laterals should have a double or twin cross-section. In ie Imperial Valley the absolute minimum quantity of water is ap-roximately 25% of the absolute maximum—a very unusual condition

of affairs. If the sections are identical, it is possible to use one canal for a long time, and have it never less than one-half full, and consequently the velocity of the water is never low enough to deposit the finer silt. This allows a sufficient time for cleaning one canal and then the other every year. When the demand increases beyond the capacity of one canal, both channels are more than half full. In this region, therefore, it is possible with the twin section to control the velocity between the excessive silt-depositing lower limit and the bank-erosion upper limit with absolute certainty. The additional construction expense of the canals for the twin section is much more than justified by the very greatly reduced maintenance charges thereafter.

Duty of Water.—As a basis for estimating the quantity of water required under the Yuma Project, investigations were made, under the direction of the University of Arizona Experiment Station, to determine the water required for various crops in the Yuma Valley. As a result of this work, it was decided that for the average 40-acre unit, 5.8 acre-ft. per annum, measured at the delivery box at one corner of the field, would be required. Such a figure is exceedingly interesting, but was not obtained under the usual operating conditions. Indeed, the satisfactory delivery of water has been made so recently and in such relatively small quantities in the Yuma Valley as not to justify any definite conclusions.

Water has been actually used in irrigation in Imperial Valley for more than 10 years, and although there seems to be no reason why the duty of water there should not be essentially the same as in the Yuma Valley, the quantity used in the former is only about half as much as indicated by the experimental work mentioned.

The crop census taken by the Zanjeros of Imperial Water Company No. 1 for that district during 1911 is as follows:

Alfalfa	44 262	acres
Barley	28 897	"
Corn	12 034	"
Cotton	6 263	"
Melons	2 153	"
Vineyards	1 352	"
Truck	1 092	"
Asparagus	192	"
Miscellaneous	3 327	"
Total	99 572	acres

To supply this acreage, the company bought from the C. D. Co. uring the period 274 665 acre-ft. of water, or an average of almost xactly 2¾ acre-ft. per acre under cultivation. This is net, after de-ucting the 10% seepage and evaporation allowance given by the C. D. o., as already explained. Of this net quantity, according to the ater company's report, 92.3% was delivered and charged to the stock-olders, making the average quantity of water used, measured at the rmers' boxes, 2.538 acre-ft.

The quantity of water used in irrigation depends on so very many fferent factors—quality of the land, nature of the crop, proper prep-ration and leveling of the land, and time of irrigation—that it is only y such general figures covering large areas that much tangible in-ormation for engineers is obtained. It must be remembered, however, hat the water supplied is charged for on a quantity basis, which ndoubtedly tends to minimize the quantity used, as well as the fact hat the farmers know they can have all the water they want at any me they want it, every day in the year. On the other hand, water sers—stockholders—are charged their *pro rata* of maintenance and peration expenses, regardless of whether or not they use any water, nd also for a minimum of 1 acre-ft. per share of stock. Additional ater is 50 cents per acre-ft.

The use of water is increasing in this district, and is in large easure due to the increasing acreage in alfalfa and cotton. The gures are:

1909	214 333	acre-ft., net.
1910	236 631	" " "
1911	274 665	" " "
1912 (Estimated)	320 000	" " "

The figures for other districts are not available, but are probably milar.

Litigation.

The Salt Works Suit.—The New Liverpool Salt Company, whose roperty was inundated and destroyed on March 8th, 1905, began suit or damages to the extent of $180 000 for land and salt deposits and 30 000 for plant, changing the figures to $325 000 and $75 000, respect-vely, when the destruction became complete. In July, 1906, a compro-ise was suggested on the basis of $50 000 cash, but the management of

the C. D. Co. declined to consider it. On January 10th, 1908, the case was decided, awarding the Salt Company $456 746.23 damages and $1 500 costs, and a permanent injunction was issued restraining the irrigation companies from diverting more water from the Colorado than would supply the substantial needs of the people residing in the valley. Later, the United States Supreme Court affirmed the decision.

Actions of the Southern Pacific Company.—When this adverse and excessive judgment had been rendered, it was seen that the United States Courts would hold the C. D. Co. liable for all damages caused by the diversion of the river, regardless of the fact that it had occurred in Mexico. All personal property and unsold water stock, therefore, was turned over to the Southern Pacific Company at fair prices, and future payments for water rentals were assigned to the Southern Pacific Company until the moneys loaned by it should have been repaid. At the same time, suit was brought in the United States Court for damages sustained in America, and suit was filed in the Mexican Courts for damages sustained in both Mexico and the United States, and all real property in the respective countries was attached. The suits in the United States are still pending. Judgment was rendered in the Mexican Courts for $900 000, gold, against the Mexican Co., and enough property of that company was ordered to be sold to satisfy this judgment. Another Mexican corporation was formed by the Harriman interests, and permission to hold the concession of the original Mexican Co. was obtained from the Mexican authorities. At the sale, held on January 28th, 1911, this new Mexican Company bid in all the real and personal property of the Mexican Co., including the concessions from the Mexican Government, for the sum of $325 000, gold, which was less than 40% of the judgment. Thus nothing now remains of the original Mexican Co. except the organization, with a $575 000, gold, judgment against it, and additional suits aggregating nearly $2 000 000 in the Mexican Courts, and absolutely no property.

The new Mexican Corporation, called the Lower California Land and Water Company, *Compañia de Terreños y Aguas de la Baja California, Sociedad Anonima,* owns practically all the parent irrigation company's holdings in Imperial Valley having any value, but has not yet taken possession. Shortly after the sale by Court decree, fraudulent dealing was alleged, and on November 18th, 1911, it was advertised that the Judge of the First Instance at Mexicali would hear any and all

complaints in the matter. No one appeared, and it seems probable that the validity of the sale must therefore be confirmed. In that event, the new company will be free of any contracts with the mutual water companies, the C. D. Co., or any one else, and is probably quite beyond the reach of the American Courts. This, however, means little to the water users living in either the United States or Mexico, as the Mexican Government has issued rules and regulations by which water must be sold under the Mexican concession, and these fix the price at 50 cents per acre-ft., and practically in no wise affect the conditions under which American users now receive water.

The C. D. Co., in that event, would be a mere shell; it owns only 65 miles of main canals which produce no revenue whatever, cost no little to maintain, and are hence liabilities instead of assets; it also owns its office, grounds, and buildings, all of which are under attachment, and its liabilities exceed $2 000 000.

Appointment of Receiver.—On December 16th, 1909, the Title Insurance and Trust Company, trustees for the bond issue of the C. D. Co., applied to the Superior Court of Imperial County to declare the C. D. Co. insolvent and appoint a Receiver, which application was granted. The Southern Pacific Company has bought approximately $325 000 worth of Receiver's certificates, which, together with the major portion of the water rentals received from the mutual water companies, has kept the property going. Application has been made to sell the property, but this has been delayed as long as possible by the attorneys representing the bondholders, the New Liverpool Salt Company, and the old stockholders of the C. D. Co. In a few months, however, it seems probable that this will be accomplished.

Formation of Imperial Irrigation District.—Because of the various difficulties and the serious litigation, the people of Imperial Valley, on July 14th, 1911, by a vote of 1 304 to 360, elected to form the Imperial Irrigation District. According to the present law of California, this district can condemn property, even of a public service corporation, and all taxable property within the district is assessable for its needs. It is authorized to incur a bond issue of 50% of the assessed valuation of the property of the district, and the five directors are elected by all voters in the district just as in the case of county and State officials. The assessed valuation of Imperial County this year is $16 161 923; the value of its products is $10 000 000.

It is intended to acquire all the property of the C. D. Co. and the main canals and works in Mexico, giving bonds of the Imperial Irrigation District in exchange therefor. It has not been decided definitely whether the mutual water companies are to be retained, or whether the district is to own and control the entire water system—probably the former will be done.

The present law of California, under which this district was formed, is extremely interesting to water supply and irrigation engineers. It is a considerably changed form of the Wright Irrigation Act, under which, some 20 years ago, a number of irrigation districts were created in California, all of which resulted disastrously. It is believed that in its present form the law is a practicable one, and experience with it will be awaited with much interest.

The Abejas Diversion.

The excessive overbank flow during the summer flood of 1907 started cutting back fingers, as has already been stated. The flood of 1908 continued the work, and made it evident that the deep finger which first would have its grade receded to and through the bank of the Colorado, and thus again divert the entire river to the west, was one of the feeders of the Abejas, and that such diversion would occur about 20 miles below the International Boundary Line.

The situation was carefully watched, and the various interests affected were fully advised of developments. The United States Government had taken no tangible step to repay the moneys expended in closing the second break and in subsequent levee protection work, nor anything definite whatsoever with the Mexican Government looking toward a joint and satisfactory control of the situation. All interests, nevertheless, seemed to feel that the Southern Pacific Company would advance funds to protect the valley when a critical stage should be reached. The writer, in local charge of the situation for that company, had become fully convinced that the truest and best interests of all concerned would no longer be served by the railroad company standing in the breach, and recommended doing absolutely nothing further in protecting the valley than to maintain the existing levee system. In almost everything there comes a time to decline longer to carry the entire load. This recommendation was approved by the higher officials. When the summer flood of 1909 had passed, the

expected diversion was an accomplished fact, and as a result the entire low-water flow followed through the Abejas, spread out in a wide sheet without any defined channels, gathered into Volcano Lake and Hardy's Colorado and thence reached the Gulf.

The water in the river at the break dropped somewhat, and, as the river fell to its low-water stage, the water surface for a given discharge was found to be unusually low. The reasons for this have already been explained, but were not then fully understood. The demand for water in the valley increases greatly late in January, on account of the barley crop, and a serious water shortage seemed very probable.

Submerged Weir.—On urgent representations to the War Department, backed by the recommendation of the Reclamation Service engineers, permission was granted in March, 1910, to build a temporary obstruction in the river opposite the concrete head-gate, in order to raise the water a few feet and increase the flow in the canal. Work was started, but the river began rising and rendered it temporarily unnecessary. When the summer flood receded, in July, the situation was again critical, due to the large requirements in the valley, and work was resumed.

A trestle consisting of 4-pile piers, 15 ft. from center to center, was driven across the river at an angle of about 70 degrees. On this a railroad track was laid, and a little brush and considerable rock was dumped therefrom. Of course, there was no difficulty in developing a head of 2½ ft. This weir or obstruction prevented any danger of water shortage in the valley, but, not being square across the river, it produced eddy currents, just below it on the Arizona side, which cut away the bank to some extent and necessitated considerable expense in bank protection work.

The permission of the Government was given for this construction, on the understanding that it was to be temporary, and would be removed before the next spring flood. In March, 1911, all the piling was blown off, not pulled.

Late in 1910, arrangements were made to obtain the large suction dredge, *Imperial,* and with it in service no further immediate difficulty in diverting sufficient water is anticipated. (See page 1283.)

A very important fact, however, was developed in the construction and attempted removal of this weir, namely, that the small quantity

of rock dumped from the trestles which was required to raise a head of 2½ ft. at low-water stages—about 10 000 sec-ft.—was not undermined and did not settle except to a slight extent in a few places, with the summer floods of 1911 and 1912. These floods were ordinarily large, and, passing over it, had little effect in taking it away. This result was surprising, even to the proponents of rock fill methods of building weirs. The length of time it finally requires to eliminate all effects of this weir from the river flow at that point should be kept track of and reported to the Profession from time to time.

All Parties Frightened.—All the interests in jeopardy were now thoroughly frightened. It was finally realized that the Southern Pacific Company would no longer supply money for river control, and that the diversion was not only an accepted fact, but that the prophesied lowering of the river at the concrete head-gate had taken place. The fear had been that the bed of the river would be lowered at the Abejas break approximately from 5 to 8 ft. and that this lowering would rapidly run back up the river and have such an effect opposite the concrete head-gate as to prevent diverting enough water to supply the needs of the valley. This fear is now known to have been in large measure unfounded, as the permanent lowering of the water surface at the Abejas probably has not been more than 3 ft., and opposite the concrete head-gate not more than 2 ft., if indeed that much at either place.

Another fear was that the Colorado would now discharge directly into Volcano Lake, and during severe flood periods raise this body of water so high that it would flow northward and into the New River channel, thus cutting back a connection, permitting the river again to reach the Salton Sea, but by a course approximately 40 miles longer.

As the summer flood came on, the overflow covering the low lands on each side of the Abejas, especially in the vicinity of Campo Lino, was higher than any existing marks on trees, etc. To prevent this water from getting over the low divide to the north and thence to the Salton Basin, disconnected portions of the remaining gap in the levee line were partly built, the C. D. Co. through its Receiver paying the bills. While the work was in progress, the water, for long stretches, came within a few inches of the top of the fill being thrown up, and was held south of the divide only by strenuous efforts. Probably no

ery serious results would have followed in any event, although the New River flume of the West Side Main might have been damaged.

The people of Imperial Valley were now thoroughly frightened, and rgent applications were rushed to President Taft, in which the civic nd commercial bodies of California, especially Southern California, nd the State officials joined. These applications pointed out the nability of American interests in jeopardy to handle a menace orig-nating in Mexico, and the injustice they suffered.

Congress Appropriates $1 000 000.

In response to these applications, President Taft sent a special nessage to Congress, and on the eve of adjournment the two branches f Congress joined in a resolution, approved June 25th, 1910, pro-iding:

"That the sum of $1 000 000, or so much thereof as shall be neces-ary, is hereby appropriated out of any money in the Treasury not otherwise appropriated, to be expended by the President, for the pur-ose of protecting the lands and property in the Imperial Valley and lsewhere along the Colorado River, within the limits of the United States, and the President is authorized to expend any portion of such noney within the limit of the Republic of Mexico as he may deem roper, in accordance with such agreements, for the purpose, as he may nake with the Republic of Mexico."

On June 8th, 1910, the acting Secretary of the Interior, Mr. Frank Pierce, addressed to the President a communication based on informa-ion and recommendations furnished by Mr. Hill, Supervising En-gineer of the United States Reclamation Service, advising as follows:

"The ascertainment of what is necessary to be done for the purpose of accomplishing permanent avoidance of these recurring menaces to life and property on both sides of the International Boundary Line will require a thorough examination of physical conditions which, to be effective, should have the co-operation of both governments, and will consume considerable time. In the meantime, unless prompt relief is afforded, a water shortage, if not famine, is probable in the Imperial Valley within the next two months.

"In a country where the heat reaches an intensity of 120° and even higher, the great loss of property and the menace to both animal nd human life which may result, should such a catastrophe occur, renders it imperative that prompt measures be taken toward averting he same. To that end, I respectfully recommend that you designate

an engineer having familiarity with problems involving river control to proceed immediately with an examination for the purpose of determining whether such emergency exists, and if so, to take the steps necessary to avoid the same."

On the recommendation of Gen. W. L. Marshall, Consulting Engineer to the Secretary of the Interior, John A. Ockerson, President, Am. Soc. C. E., and for many years Member of the Mississippi River Commission, was, on July 19th, 1910, appointed by President Taft. He at once went to Yuma, arriving there on July 30th, by which time Mr. F. L. Sellew, Engineer of the Yuma Project, on Mr. Ockerson's request, had made a survey covering the immediate vicinity of the C. D. Co.'s intake, and had prepared a plat which showed clearly that the bed of the river was above the bottom of the head-gates and that the deficiency of water in the canal was due mainly to the silting up of the intake above and the canal below the concrete head-gate.

There were two possibilities: one was to get dredging machinery into place as rapidly as possible and dredge out the canal; the other was to raise the water in the river by a submerged dam, the latter being temporarily necessary because of the time required to build dredges to do the necessary work. As explained, the weir was begun in the latter part of July and completed within a month, and the contract was not let for an efficient suction dredge until late in December, the delays in starting the latter being in part due to difficulty in arranging funds therefor, and in part due to a belief in some quarters that, with the submerged weir in place, a dredge was unnecessary.

Mr. Ockerson made an inspection of the Imperial Valley during the latter part of August, and about the middle of that month put out a surveying party which ran a stadia line along the Colorado River from about 6 miles below the Boundary Line to the Abejas break; thence down the dry bed of the Colorado for about 25 miles; down the Abejas River from the point of diversion about 5 miles; and made several cross-sections of the Abejas for the purpose of selecting the best site for a rock fill barrier dam—all of which required about 6 weeks.

On October 4th, Mr. Ockerson reported to the Secretary of the Interior that the Imperial Valley would never be safe from the menace of western diversions due to flood-waters until ample works were con-

structed to confine such flood-waters to narrow limits along the river proper; that there had been no appreciable lowering of the river bed on account of the Abejas break; that in extreme cases diversions to the west might depress the low-water plane opposite the concrete headgate and render it difficult to supply water to the Imperial Valley with the present diverting works; that the maintenance of levees consists not only in keeping up the cross-section, freeing it from weeds, brush, and burrowing animals, but also in holding in check the tendency of the river to erode the banks and threaten to breach the levees; that a levee located 3 000 ft. from the westerly bends of the stream would probably remain intact for a long time; that if the C. D. Co. levee line had been carried down along the river instead of where it was built in 1907, it would have reached a point 6 miles below the Abejas break, and no break would have occurred there; that completing the upper levee would undoubtedly protect the Imperial Valley from floods for a short time, but constituted only a partial solution of the problem, and even that only temporarily; that, finally, the proper protection of lands in the Imperial Valley required that the Colorado River be restored to its former channel and an effective line of levees be built from a point on the existing levee system about 6 miles below the International Boundary Line and following along the west side of the stream to a distance of about 3 000 ft. from the westerly bends of the river, approximately 25 miles, where the flood height would be at an elevation below the ground line in the vicinity of Volcano Lake and any diversion of the water would not cause a tendency to flow north.

It was estimated that such a levee would require about 1 300 000 cu. yd. of earthwork and 450 acres of clearing and grubbing; and it was recommended that the top be 8 ft. wide, the slopes 3 to 1, and the berm between the toe of the slope and the edge be 40 ft. wide, and that borrow-pits be on the river side, with traverses 50 ft. wide at intervals of 400 ft.

In commenting on the situation early in September, 1910, Gen. Marshall recommended completing the northern line of levees, as originally designed by the writer, to prevent the water from getting north, and providing a suitable and practical intake for the canals of the C. D. Co. He also suggested that, if the present intake (concrete heading) be closed and the Imperial Canal be extended to the Laguna

Weir, the matter would be solved, as far as American interests were involved.

River Levee Plan Adopted.—A month later Mr. Ockerson went to Washington for a conference, the result being that the work suggested by him was ordered begun. On November 25th bids for the levee construction were opened in Yuma. According to the specifications, the contractor was to assume all risks of interruption of the work by floods, and as the season was by this time far advanced, the prices were deemed too high and endeavors were made to get the work done on force account.

At about the same time it was discovered that the Mexican Government—as would naturally be expected—could not consistently permit the United States Government, or any of its officials acting as such, to perform work on Mexican soil. For six years, the need for making satisfactory arrangements and agreements with the Mexican Government regarding the Colorado River and its control, and proper and equitable division of its waters, had been fully understood, but practically no progress had been made. That fact, however, makes it even more surprising that requests on the part of the United States should have been made to enter Mexican territory and do work therein. The difficulty was very easily overcome, of course, by operating through the medium of a Mexican corporation, as the Southern Pacific had done, and the C. M. Co., which, as has been explained, is a very large land company, was chosen. Consequently, nothing was done in the name of the United States, but the engineer in charge acted under power of attorney from the C. M. Co., there being a gentleman's agreement between the United States and the C. M. Co.

In this way, on December 12th, contracts were awarded for levee construction, the first 9 miles, aggregating 425 000 cu. yd., at from 19 to 22½ cents; the next 6 miles, aggregating 336 000 cu. yd., at 23 cents; and the remainder, aggregating 325 000 cu. yd., at 36 cents. These figures were afterward considerably increased, the total quantity being 1 277 984 cu. yd., and the total cost, including $20 000 paid for duties, $452 434. The work in the immediate vicinity of the Abejas break and the small quantity of grading on 4 or 5 miles of temporary track was done by the C. M. Co. on force account. President Lovett, of the railroad company, offered to supply, essentially at cost, all the organization, men, equipment, and supplies required for closing the break and for doing any other work that might be deemed necessary.

The Mexican Government had given assurances to the American Minister that duties on stock, material, and supplies would be remitted, which was considerably more than had been done when the railroad had the work in charge. The Government officials, however, were not satisfied with this, and thought that all material should be passed free—a matter very much more difficult to arrange under Mexican laws, as it would require Congressional action, and that country was already in the throes of a revolution. However, after a delay of 2 weeks in getting stock across the line, it was decided to arrange for the duties and depend on a refund later. The contractors began work early in January. At about the same time one pile-driver was put to work on a trestle across the Abejas, and by February 2d (river discharge, 11 000 sec-ft.) the temporary track was completed to and over the trestle, and rock dumping began.

The method was that developed in closing the first and second breaks, and used later in closing the gap in the Laguna Weir, one trestle being deemed quite sufficient, as the maximum head was not expected to exceed 7 or 8 ft. At the Andrade quarry there were approximately 15 000 cu. yd. of rock ready to load with steam shovels, and the quarry was well developed, having a face about 900 ft. long and averaging 40 ft. high. Two 4-cu. yd. steam shovels were secured from the Southern Pacific Company, and a 2½-cu. yd. shovel of the Reclamation Service was brought down from the Laguna Weir. Work trains, men, and "battleships" were obtained from the railroad, as required, and rails and track material were furnished on a rental basis.

On February 7th, a sudden rise (maximum discharge, 23 000 sec-ft.) caused a breach in the trestle; this was closed 10 days later. On March 7th another small rise in the river carried away seven bents, and 6 days later another rise (maximum discharge, 35 000 sec-ft.) brought down a mass of drift and wrecked a considerable length of the bridge, caused the loss of a pile-driver and one steel "battleship," and drowned one man. On the 28th the pile-driving was resumed and the operations were continued without further mishap until May 15th, when work on the dam was shut down.

In making this closing, the rock fill was not kept at uniform height for the entire length of the trestle, the overpour for some of the time being confined to three places with a total length of from 260 to 275 ft. It is probable that this explains in large measure the breaking of the

trestles by floods and drift, after obtaining an effective rock mattress, such as is provided by dumping two or three cuts of "battleships" all along. In building the Clarke Dam, floods (maximum discharge, 32 000 sec-ft.), with heavy drift, threw the trestle out of line in only one place after rock dumping began, and caused no other damage.

The number of cars unloaded in the Abejas closing work is given in Table 22.

TABLE 22.

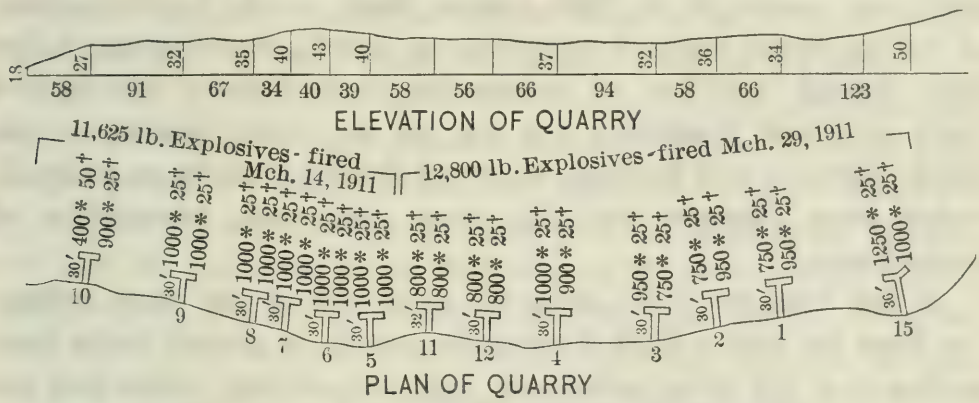
Period.	"Battleships."	Flats.	Dealey.	Dinky.
January 17–31	83	....	....	12
February 1–10	314	....	....	....
" 11–20	350	5	....	....
" 21–28	616	....	....	34
March 1–10	181	....	....	....
" 11–20	108	....	....	....
" 21–31	207	....	....	108
April 1–10	747	43	....	20
" 11–20	781	40	6	....
" 21–30	581	70	37	....
May 1–2	41	3	....	8
Totals	3 996	161	43	182

The total quantities were: 139 860 cu. yd. of "battleship" rock, 193 cu. yd. of flatcar rock, 516 cu. yd. of Declez rock, and 1 092 cu. yd. of quarry rock in dinky cars, a total of 143 400 cu. yd. up to May 2d. The total quantity of rock used to May 15th, when work closed down, was about 180 000 cu. yd. The total cost of the dam is given as $347 500.

About 140 000 cu. yd. of rock were used before the water was practically shut off in making this closing. The reasons for requiring such a large quantity probably are that relatively little flatcar rock of large size was used, the fact that the rock fill was not carried along at a uniform height for the entire length of the overpour, and the slow rate at which the rock was unloaded. The first methods of quarrying were not well adapted for giving the maximum output, consisting of operations along the top of the rock mass by the edge of the quarry face, but, later, horizontal tunnels were driven into the quarry face at intervals along the bottom and large charges of explosives were used, after which the output was much increased.

The levee work went along very rapidly, the contractors fortunately encountering no flood difficulties or delay, and on April 7th the last of the grading outfit left the work.

While operations were in progress, the Revolution in Mexico began, and resulted in the abdication of President Diaz. On February 21st, the Revolutionists captured Algodones, and took possession of a work train for half a day. On April 16th a large body of Mexican Federal troops arrived at the break and remained guarding the work from interruption until May 10th.

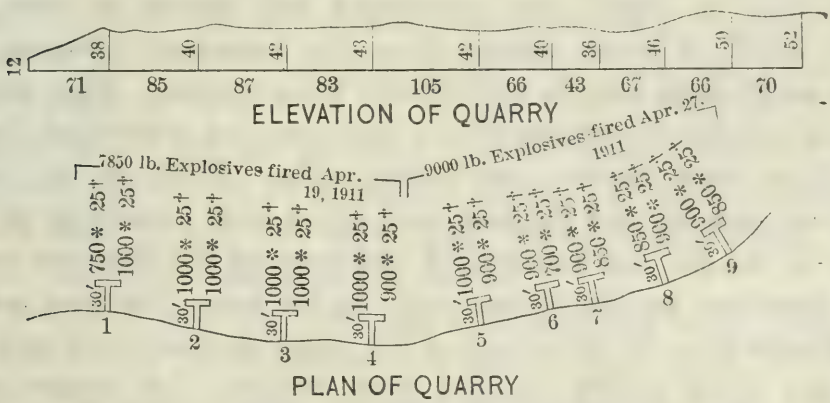


CHAMBER BLASTS AT ANDRADE—FIRST SET

Pounds, Judson Powder_____* Pounds, 40% Giant____†

Loosened up 51 600 cu. yd. Swell, 10 000 cu. yd. Material broken small

FIG. 67.


12
38 40 42 43 42 40 36 46 50 52
71 85 87 83 105 66 43 67 66 70
ELEVATION OF QUARRY
7850 lb. Explosives fired Apr. 19, 1911
9000 lb. Explosives fired Apr. 27, 1911
PLAN OF QUARRY

CHAMBER BLASTS AT ANDRADE—SECOND SET

Pounds, Judson Powder_____* Pounds, 40% Giant _____†

Loosened up 58 000 cu. yd. Swell, 13 000 cu. yd. Material broken small

FIG. 68.

Damage to the Work.—The 24.6 miles of levees were constructed in accordance with the recommendation, namely, with a top width of 8 ft., side slopes of 3 to 1, borrow-pits on the river side 400 ft. long, with intervening uncleared traverses 50 ft. wide, a berm width of 40 ft., the entire ground covered by the levee and the berm cleared, and roots and stumps grubbed, and a muck-ditch, of such depth as would

reach through the adobe soil under the construction, dug out and filled with good material under the axis of the levees. Except for a very few cases of logs and brush in the levee section, and inefficient muck-ditching reported to have been disclosed where the levees were broken, the dikes were very well constructed in accordance with the specifications. The levee was built to a grade of "5 ft. above the high-water marks of the 1909 summer flood, chiefly for the purpose of having excess material wherewith to remedy deterioration, rather than through any fear of overtopping from floods." No railroad track or gravel blanketing was put on the levees, because of the belief "that it would be better to extend the levees as far as practical, rather than dissipate available funds for mere convenience of maintenance."

A low levee was built along the south side of the break, extending from the dam a short distance up stream, to prevent water from getting into the levee borrow-pits until the grading outfits had finished work in the vicinity. As the water in the river was gradually raised, partly by increased discharge (total, 19 000 sec-ft.) and partly by increasing the height of the rock fill across the break, this low levee was, on April 21st, overtopped and almost at once about 1 000 sec-ft. of water started down the borrow-pit clearing, about 2 000 sec-ft. going into the old Colorado River channel. When the water hit the uncleared traverses it cut the berm and side-wiped the levee at almost every traverse for several miles. Work on the rock fill dam was stopped, and the men were set at work protecting the levee. Later in the day this was stopped, and work on the dam was resumed. The latter was facilitated materially by the waters breaking over into the borrow-pits, the elevations of the water surface above and below the dam being quickly changed from 79.6 and 71.6, respectively, to 78.6 and 71.0, the depth of overpour being reduced 1 ft. and the head on the structure 0.4 ft. By dumping rock and filling the holes where the confined overpour occurred, the situation there was soon in hand, and 8 days later (April 29th) the elevation of the lowest point of the dam was 80.6, or 1 ft. higher than the water surface up stream, where the water broke into the borrow-pits, and the flow over the structure was stopped.

By that date the levee to the south was cut entirely through in several places in the first 6 miles, and it was evident that the water

would soon merely detour around the dam and continue to follow the Abejas channel below. The river discharge then was 21 900 sec-ft., and about 4 000 sec-ft. were going down the old channel, the remainder running through the levee breaks and toward the west. On May 7th, the discharge of the river had increased to 32 800 sec-ft., of which perhaps 9 000 sec-ft. were running down the old channel of the Colorado, and the water varied in height from 4.3 to 6 ft. below the top of the levee in the 2 miles immediately north of the dam.

The rock fill was then up to the track all across, and the total percolation through it was reduced to about 120 sec-ft. The eddy currents below the dam, 9 days later, weakened the earth fill about 300 ft. from the north end of the dam, until the water broke through, and in a few hours the entire discharge of the river (except a little overbank flow) was going through it and down the Abejas. Soon the earth fill on the south was cut through, and thus the rock fill dam was made an island in the Abejas channel, which is the situation at present.

The final injury on the levee work prior to the summer flood was three breaks and several places side-wiped on the north levee, totaling about 16 000 cu. yd., and thirteen breaks, varying from a few hundred feet to more than 2 miles in length, and much side-wiping in the first 8 miles of the south levee, totaling about 200 000 cu. yd., or about 50% of the original earthwork in this stretch. The fact that such disaster was caused by so small a quantity of water reaching a maximum depth of only 4 ft. on the levee shows clearly the ease with which the material of the region is eroded.

The protective measures used were sand bags and brush placed to check erosion and the dynamiting of the traverses which, with the drift their vegetation caught, deflected the water to the levee section. The latter procedure had the bad effect of converting the borrow-pits into a continuous canal, but with severe eddy currents caused by the remains of the traverses. These endeavors had little effect; indeed, to hold long stretches of embankment against such action is practically impossible.

Board of Review.

On June 1st, Mr. Fisher, Secretary of the Interior, called a Board of Review consisting of Mr. F. H. Newell, Director of the U. S. Reclamation Service; Gen. W. L. Marshall, Consulting Engineer to the

Secretary of the Interior; J. B. Lippincott, M. Am. Soc. C. E., formerly a Supervising Engineer of the U. S. Reclamation Service and now an Assistant Engineer of the Los Angeles Aqueduct; Mr. C. E. Grunsky; Mr. J. A. Ockerson, Engineer in charge of the work; and Gen. Harrison Gray Otis, President of the C. M. Co., to report on the work done under the appropriation. All the members of the Board have examined the territory and understand the situation fully, although none of them except Mr. Ockerson has been on the ground since the Abejas diversion occurred.

On June 7th this Board made a report based on information as to the recent work done and results obtained, supplied by Mr. Ockerson, and answering specific questions submitted by Secretary Fisher. The full text of this report is as follows:

"1.—A breach in the west bank or levee of the Colorado River, if made at or within a mile south of the California boundary, or south of Mile 18, below Yuma Bridge, will result in water flowing directly into the drainage areas of the Alamo and New Rivers and thence into Salton Sea, which would be disastrous to property in the United States.

"2.—A breach in the river bank at any point between Miles 18 and 55, below Yuma Bridge, will result in spreading water over the Delta of the Colorado River, with a flow into New River *via* Volcano Lake, menacing Imperial Valley.

"3.—(*a*) The best practical method for the protection of land and property in the United States against a discharge directly into the Imperial Canal and thence through Imperial Valley into Salton Sea, is to protect and maintain the levees as at present located for a distance of at least 10 miles south of the California boundary and to hold the river by adequate bank revetment practically on its present alignment. (*b*) This levee, if extended to a point opposite the south boundary of Arizona, or about Mile 27, will also prevent a discharge directly into the Paredones.

"4.—(*a*) As a remedial or precautionary work to prevent damage which might result from a crevasse directly into the Imperial Canal or Alamo River, we have considered a secondary levee west of the river levees, across Imperial Canal, and large channels leading to the natural depressions, or diverting works, conducting the water southwesterly into channels leading into Volcano Lake, but it is believed that the cost of any such secondary defence could be better expended in maintaining the main line of defence at the river. (*b*) As a necessary defence against the northerly flow of any water reaching Volcano Lake, whatever be the treatment of the Lower Colorado River, there should be an embankment well protected against wave wash on its

south slope, constructed about on the line of the levees already built extending from high ground north of Volcano Lake to a connection with the levees already built by the California Development Company, southwesterly, toward this region from the Colorado River. The top of this embankment should at its western end be not less than 10 ft. higher than the rim land at Volcano Lake. This embankment is an essential requisite as a protection of Imperial Valley, against menace from the south, and should be constructed without delay.

"5.—(*a*) The maintenance of the works constructed in 1906 and 1907, closing the breaks of the Colorado into the Alamo, and the maintenance of these and of the river levees since constructed as far south as the head of the Abejas are essential requirements. Suitable arrangements for their repair and maintenance should be made with Mexico through the proper authorities. We do not consider the immediate closure of the break into the Abejas and the reconstruction of the levees below the break as essential to the protection of property in the United States. The ultimate treatment of this section of the Colorado River in co-operation with Mexico may well be determined by negotiations between the governments of the two countries. As a feature of the permanent solution of the river problem it is desirable that the Abejas break be closed, that the levee constructed in 1911 be repaired and maintained, and the Colorado River restored to its former course. (*b*) Provided the water of the Colorado is discharged into the Gulf of California through the Abejas into the Pescadero and Hardy Rivers, there is little probability of the cut back affecting the Laguna Dam. Such cut back will not injuriously affect the heading of the Imperial Canal or levees adjacent thereto, with a possible exception of requiring the lowering of the intake of the Imperial Canal a few feet. This is not a serious matter, and is one that should be dealt with by the California Development Company itself when necessary. The diversion by the California Development Company should be facilitated during low-water stages by dredging, or by lowering the sill of its intake, rather than by placing obstructions in the channel of the river below the intake.

"In view of the existing emergencies along the Colorado River, arrangements should be made with the government of Mexico to provide for the early creation of an International Colorado River Commission, embracing in its membership both American and Mexican engineers, invested with large powers and ample authority to examine into and to submit a basis for the adjustment of all questions relating to the conservation, use, and control of the waters of the Colorado River with a view to such governmental action as shall result in a complete, just, and final settlement of all such matters at issue between the two nations. We recommend that further work should be undertaken at once and in approximately the following order:

"(*a*) The levees north of Volcano Lake should be raised, strengthened, and extended.

"(*b*) The existing levees along the west bank of the Colorado River to the Abejas should be repaired and protected. For this purpose and to meet emergencies, there should be immediately available the sum of at least $1 000 000. This sum provides only for the temporary maintenance of levees, and does not include the systematic revetment of the river banks.

"The conference ventures to suggest certain international questions which are involved and which will inevitably have to be met sooner or later:

"(*a*) The matter of the permanent protection of existing works on both sides of the international boundary line, the construction of further works, and the conditions under which the present and future projects may be carried out on Mexican soil, with the consent and co-operation of the government of Mexico for the benefit of both countries, to the end that the greatest practicable quantity of water of the Colorado River may be made available for irrigation by means of storage reservoirs and otherwise, and the least possible quantity be permitted to flow unused to the sea, and to what extent the cost of such maintenance should be chargeable to properties benefited and to what extent chargeable to either government.

"(*b*) That permanent agreements with the government of Mexico shall be entered into, having in view the just apportionment of the waters between the two countries, irrigation to be paramount to navigation.

"(*c*) The method by which either nation may acquire rights of way for canals, levees, and related works, each within the territory of the other, and the authority to maintain such works.

"(*d*) The modification of the boundary line between the United States and Mexico with a view to facilitating the solution of the entire Colorado River problem. An authoritative, just and final determination of this important question, now a matter of public discussion, will have the effect of removing existing doubts in the public mind and of settling the matter for the benefit of all concerned.

"The members of the conference desire to call attention to the fact that the plan and execution of the work accomplished during 1911 followed well-established principles of good engineering. That so large an amount was accomplished in such a brief space of time, under adverse circumstances, is worthy of the highest commendation. That the restoration of the Colorado River to its former channel was not realized is chargeable to the delay in the negotiations, which prevented prompt inauguration of the work and the prosecution during the low-water season, and also to the disturbed political situation and strike which demoralized labor conditions. The members of the conference,

in addition to the conclusions above reached, present also a statement of physical and related facts embodied in an abstract of the data available, and found largely in the reports of J. A. Ockerson and of C. E. Grunsky. Also in the printed hearings before a subcommittee of the Senate Committee on Claims, referring to Senate 4170, January 21, 1909."

When President Taft received the report, he sent a special message to Congress recommending the appropriation of another $1 000 000 to continue the control work, but Congress failed to comply. It is desired to call particular attention to the last part of this report containing Suggestions *a, b, c,* and *d* regarding certain international questions involved, which sooner or later must be met. These suggestions contain the crux of the entire situation, and it is to be hoped that they will be expeditiously followed out. It is perhaps desirable to call attention to the wording of Suggestion *d* regarding modification of the Boundary Line between the United States and Mexico, the suggestion being implied that the matter be taken up more with a view of putting a quietus to the proposition, than with the idea of obtaining any territory from the Mexican Government.

Surveys by the New Mexican Company.

As soon as the summer flood of 1911 began to recede, and the extent of the damage sustained by the new work was ascertained, the Harriman interests controlling the new Mexican Company deemed it wise to ascertain how effective the line of levees north of Volcano Lake would be in holding back the waters of a very large summer flood. Accordingly, three surveying parties were put out early in July under the immediate direction of Mr. Hind. The surveys were carried down the river to The Colony and as far west as Volcano Lake, much of the territory being covered quite thoroughly. The field work was completed during the latter part of September, and the data were assembled and mapped. Mr. Randolph analyzed these data and compiled a report which was forwarded to Mr. R. S. Lovett, Chairman of the Executive Board in New York City, recommending:

1st.—That the westerly portion of the Volcano Lake levee be raised and extended so as to occupy a plane 6 ft. above the then present crest; and

2d.—That an effective levee system, including a rock dam shutting off the Abejas diversion, be constructed along the river to a distance

of 16 miles below such dam; that the entire levee should be blanketed with gravel; and that a railway track be laid thereon in good condition for further operation when necessary.

Mr. Lovett, on December 13th, 1911, made a formal offer to President Taft, on behalf of the Southern Pacific Company, to return the Colorado River to its original channel and to maintain the levees necessary to keep it there for one year, providing the Southern Pacific Company be repaid for the work done in 1906-07 at President Roosevelt's request, in the sum of $1 083 673.97, being the amount reported as proper under the circumstances by Mr. Grunsky in his statement to the Committee on Claims during 1908; and provided, further, that an additional appropriation of $1 500 000 be placed at the disposal of the President of the United States with which to pay the actual cost of the work to be done and the cost of maintaining it for one year, the Southern Pacific Company to stand any excess of cost over and above such an amount; that the transportation charges of the Southern Pacific Company in connection with the work should be in accordance with the arrangements in effect during the work under Mr. Ockerson's direction; and providing, further, that should the Southern Pacific Company fail to return the Colorado River to its former channel and to retain the levee for one year thereafter, then in that event the Southern Pacific Company should receive no compensation or reimbursement for the work which it would do under that offer.

This proposition was referred to Gen. Marshall, Consulting Engineer for the Department of the Interior, who, under date of January 5th, 1912, reported in substance that the work proposed should not be done either by it or by any other agency on behalf of the United States at this time, nor until the entire subject of improving the Colorado River and utilizing its water should be investigated by an International Committee representing both the United States and Mexico. The following day, the Secretary of the Interior, W. L. Fisher, forwarded Gen. Marshall's report to President Taft and approved of its conclusions, stating that the suggestions constituted the most important recommendation of the Advisory Board of June 7th, 1911, and adding:

"I consider it of great importance that negotiations should be immediately opened and vigorously conducted with a view of arriving at a treaty with Mexico covering this subject."

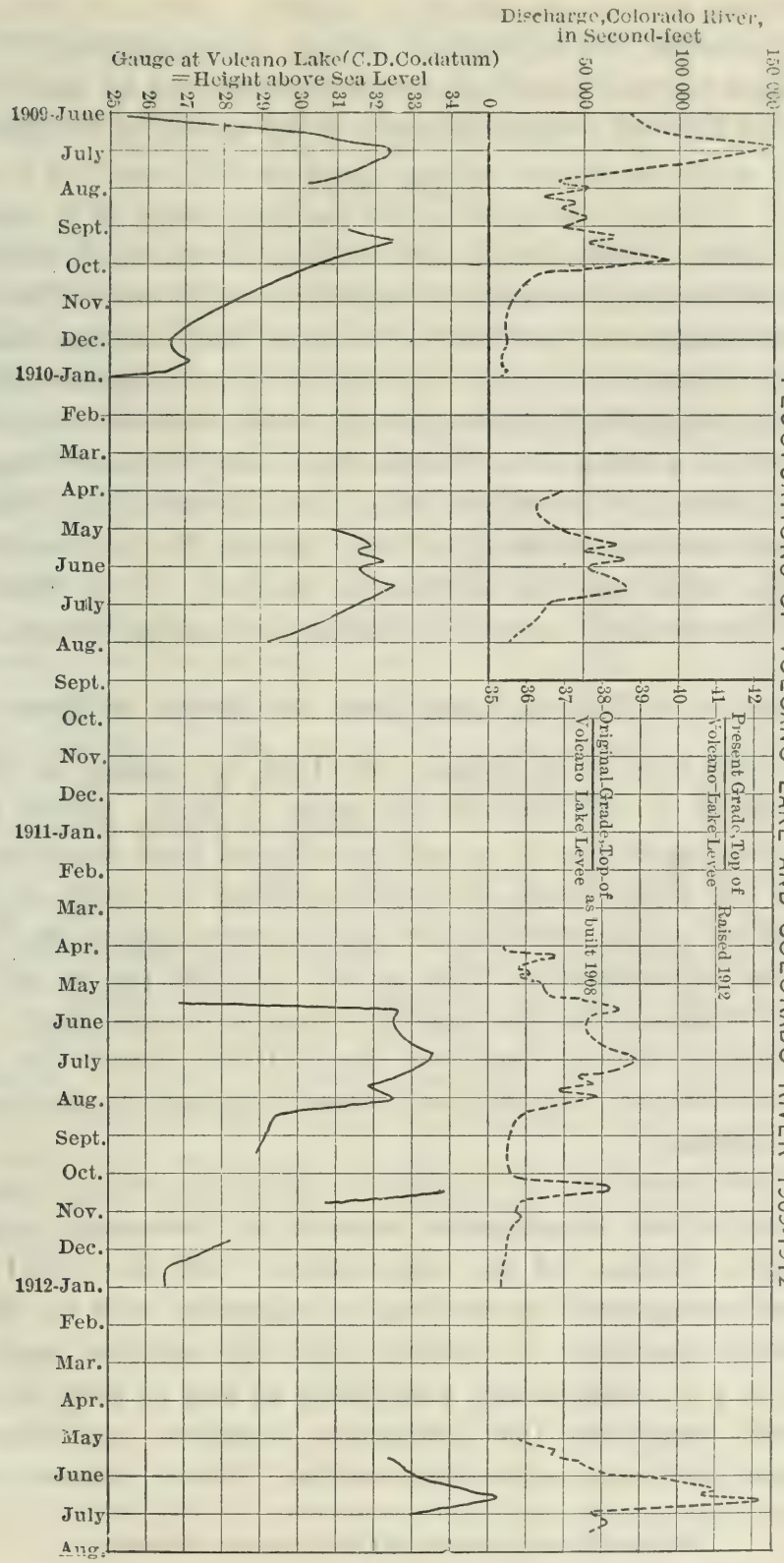

FLUCTUATIONS OF VOLCANO LAKE AND COLORADO RIVER 1909-1912
Discharge, Colorado River, in Second-feet
0
50 000
100 000
150 000
Gauge at Volcano Lake (C.D.Co. datum) = Height above Sea Level
25
26
27
28
29
30
31
32
33
34
35
36
37
38
39
40
41
42
Present Grade Top of Volcano Lake Levee Raised 1912
Original Grade Top of Volcano Lake Levee as built 1908
1909-June
July
Aug.
Sept.
Oct.
Nov.
Dec.
1910-Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
1911-Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
1912-Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.

FIG. 69.

President Taft, in his message to Congress a few days later, placed the whole matter before Congress without recommendation.*

Perhaps the most important information obtained by these surveys is the fact that the average elevation of the bottom of Volcano Lake is 28 ft. while the general average prior to 1907 was 17.8 ft. above sea level. In other words, the bottom has been raised 10 ft. since the Colorado began to flow directly into the lake, during which time it has discharged into it approximately 30 000 000 acre-ft. of water. The streams below, draining to the Gulf, are now normally clear, showing that most of the silt content of the water reaching the lake is being dropped over its bottom, together with some material eroded from the new channels formed by the diverted river. On the other hand, very much suspended material carried by the waters is let down before reaching the lake. The conditions are extreme, yet they indicate the extreme silt deterioration which may be expected in reservoirs on the Gila, Salt, Verde, Colorado River below The Needles, and similar streams.

Improvement of Volcano Lake Levee and Repair of River Levee.

Contracts were let on January 4th, 1912, for raising the Volcano Lake levee 3½ ft. and widening the crown to 12 ft., 155 000 cu. yd. at 25 cents; repairing the recently constructed levee along the river north of the Abejas break, the contract calling for 15 000 cu. yd. at 22 cents; and paving with rock the south or water face of the Volcano Lake levee, approximately 70 000 cu. yd., for $1.50 per cu. yd. This rock was obtained from the mountain sides at or near Cerro Prieto. The temporary railroad track from the C. D. Co. levee to the Abejas break was taken up, and the track material returned to the Southern Pacific Company.

Assistant Secretary of the Interior Thompson went to the City of Mexico to make arrangements whereby the Government might do this work in Mexico, but was unsuccessful. Until the United States concludes arrangements for working in conjunction with the Mexican Government, operations on Mexican soil will doubtless have to be handled in a roundabout way, particularly as long as there is fear of unfounded complaints that lawlessness interferes and delays the progress of such work as may be permitted. Mexico, however, author-

* See also House Document 204, 62d Congress, 2d Session.

zed the foregoing work to be done by the C. M. Co. through the engi-neer assigned by the United States to do the work, and a gentleman's greement was reached between that company and the United States.

Conclusion.

Because of the various successful and unsuccessful work done in he region, the engineering features of irrigation and river control long the Lower Colorado are now understood, and engineering con-truction methods are thoroughly developed. The successful attempts n closing breaks along the river with rock fill barrier dams according o the method developed during the first and second closings have tandardized this class of work. The Southern Pacific Company can asily, on very short notice, furnish all the equipment, men, and rganization needed to do all the various classes of work involved, irectly or indirectly, in controlling the river. The essential features f successful levee construction there have been made very clear. The naintenance and operation of the irrigation canals involve caring for xcessive quantities of silt, and sufficient data regarding the silt prob-em in the Main Canal have not yet been obtained to decide on the ost economical method of diversion.

The Colorado River Delta now presents no unusual, unsolved en-ineering difficulties; its problems are chiefly matters of statecraft, n both river control and irrigation. At the conclusion—in the near uture—of existing litigation in American Imperial Valley, irrigation f the territory will be notably free from legal and managerial en-nglements. This will be as soon as, and not until, reasonable treaty rovisions between the two nations are arranged. Such a treaty is in-ispensable for the proper handling of river control work. Fortunately, oth Governments profess not only willingness, but impatience, to ad-ust the matter.

DISCUSSION

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This interesting paper furnishes valuable information relating to the troubles of the irrigation engineer and the irrigators. A record of the memorable struggles with flood-waters in the Colorado Delta in recent years will always be a source of valuable information.

The rich fertilizing qualities of the silty waters of the Colorado River are well known. The important point to keep in mind is the fact that the rich and desirable sediment always comes down with the first rising river waters, and the undesirable barren sandy silt during high water and the falling river stages. It follows, therefore, that the desirable silt is to be had largely from the rising river waters, and that suitable management and wise control of the scouring sluices and regulating gates at each end of the Laguna Weir are of the greatest importance in the maintenance of the entire canal system. In order to reduce silting troubles to a minimum, the mean velocity of flow throughout the entire canal system should be as nearly uniform as practicable.

For this purpose the formula given by Mr. R. G. Kennedy,* is very useful, namely:

$$\text{Velocity} = 0.84\, d^{0.64},$$

hence, comparing one channel flow with another, we have practically the ratio:

$$v : v_1 : : \sqrt[3]{d^2} : \sqrt[3]{d_1^{\,2}}$$

but, after all, the main feature to keep constantly in mind is that the mean velocity varies as $\sqrt{R\ S}$, and that S is the slope of the surface of the water and not the slope of the bed of the canal. With the best of management, however, there will always be some silting up, but the silt ought to be largely of valuable character, and when dredged and deposited on the bank adjoining the county roads the farmers will eagerly cart it away for fertilizing purposes. The clean, barren, sandy silts which are so undesirable, should not be allowed to enter the head-works of the canal to any large extent. The cost of the Laguna Weir, $434 per lin. ft., when compared with the cost of the Kistna Anicut, in India, $104 per lin. ft., seems to be unreasonable, but such comparisons are always odious, especially when all circumstances are not known. The annual revenues from the Kistna system amount to $1 500 000, and if the Laguna Weir system approximates this figure in the course of events, all will be satisfied.

The author's remarks about a "rating curve" are commendable. No official record is so absolutely unreliable as the conventional rating curve for any given gauging station. It is worse than no record

* *Minutes of Proceedings*, Inst. C. E., Vol. CXIX, p. 281.

whatever, inasmuch as it is a worthless official record which will have to be officially disproved in Court before one can get authority to proceed. Experience everywhere on sedimentary rivers shows that all the cross-sections of a rising river are enormously increased in area by the scouring out effect of the down-rushing flood-waters laden only with lighter sediment. The heavy sandy sediments gradually crawl down river much more slowly than the rushing flood-waters, hence they do not show until the river begins to fall. It follows that all the cross-sections of a falling river diminish rapidly in area and return gradually to the usual low-water cross-section. These being the facts on every flood, how is it possible for any sane man to get up a rating curve which has any practical value whatever? This is particularly true in the case of a gauging station on the main river just above a large tributary stream. Mr. Le Conte.

When the tributary stream is in high flood, the water in the main river is not only high, but the current is often running up stream, which potent fact alone destroys completely the value of the rating curve.

The author's proposed plan of building a wide rock fill dam, without any concrete cross-walls whatever, seems to be plausible, in view of the valuable experience in closing the big breaks in the Colorado River banks leading down to the Salton Sea.

These rock fill dams have now stood the ravages of severe floods for the past 6 or 7 years, and are still intact and water-tight, for all practical purposes. The main object of the concrete cross-walls in the Indian dams is to shut off leakage through the broken-stone fill and hold the stones together. As the low-water flow of the Colorado River is small (6 000 cu. ft. per sec.), it becomes very important to hold all the water possible. If a successful, wide, water-tight dam can be made, as described, without them, all well and good. The writer is of the opinion that it is well worth trying, from a financial point of view, particularly at such a site as the Laguna Weir, where the slope per mile is only 1.2 ft.

In India, the anicuts are generally built where the slope per mile is 3.5 ft. and the river bed is pure sand of unlimited depth.

Morris Knowles, M. Am. Soc. C. E. (by letter).—This excellent and exhaustive paper is a most fascinating story of the ever-changing Colorado. The reader can almost picture himself on its banks, witnessing the frequent breaks, overlooking the vast overflows, and taking part in the repeated attempts at closure. The Engineering Profession can indeed feel gratified that it has members who will patiently chronicle events, analyze situations, and faithfully tell of failures. Mr. Knowles.

Much is related of irrigation needs, local causes and effects of floods, notable works built and projected for the former, the great emergency construction to overcome the latter, and studies and plans for future

Mr. Knowles.

protection and control. Little has been said, however, of the possibilities of prevention, of great works that might be built to hold back the flood flows and let out at other times the increased quantities that would make for greater irrigation possibilities, increased navigation stages, if otherwise possible, more water for power, etc.

Within a short time, and brought to a focus by the work of the Pittsburgh Flood Commission, the country has realized that we can no longer be indifferent to the vast and universal country-wide problems of regulating the flow of our streams. Without going into details of the many correlated benefits to be obtained from storage of flood flows, the devastations of the Ohio, Missouri, and Mississippi Valleys, those of the Southern Appalachian regions, as well as those of the Pacific Coast and great Colorado, so vividly portrayed in this paper, have awakened people, all over this land, to the fact that here is an important problem to solve, in which the factors have some relation, wherever we go, and from the solving of which, much other good will come.

That this is also true of the Colorado Basin is evident from the author's remarks on pages 1206 and 1208 and from Table 2. It would be of still further interest, therefore, to know whether the storage of approximately 10 000 000 acre-ft. exhausts the possibilities on the Upper Colorado, and what effect such impounding at several selected points would have in decreasing the flood discharge and increasing the low-water flow. It is possible that it would not apparently diminish the interest and intensity of the problem of protection below Yuma, on account of the peculiar local characteristics, yet it is probable that the cost of such works would be less and safe results more certain.

If data are available to show the effects of such storage of flood peaks, as is illustrated by the Pittsburgh report, it is hoped that the author will present them in his closure. If not now obtained, perhaps this discussion may call attention to the need, and emphasize the fact that here is another place and a further reason for the Nation, by its Central Government, to take hold of its water problems as a great national question, and to study, plan, and treat each group of streams as a unit from source to mouth.

Mr. Sellew.

Francis L. Sellew, M. Am. Soc. C. E. (by letter).—Turning the Colorado out of Salton Sea was not so much an engineering triumph as a triumph of engineers, of red-blooded fighters, who did not know when they were whipped, but realizing that the end justified the means, fought on to victory unmindful of its cost. All honor to them. The writer will go to the limit in expressing appreciation of results attained in the face of odds as great as any with which river engineers may expect to be confronted; but when the closing of this crevasse, which, from the nature of things, had to be accomplished by rough and ready

methods, is heralded as a solution of all engineering problems pertaining to the Lower Colorado, the writer's protest is immediately entered. Mr. Sellew.

Having been an innocent bystander while some of the work described was in progress, this paper has been read with much interest. The record of the operations incident to the return of the river to its former channel is extremely complete, although partly obscured by a mass of legal and financial entanglements which properly have no place in a purely engineering article.

Lack of time will confine the writer's discussion to an examination of some of the more important points. The data submitted in regard to the discharge of the stream and the degree of reservoir control possible vary materially from the writer's ideas on these subjects. It is also doubtful if the control attainable by reservoirs will greatly reduce bank caving, which is the real problem along this river. Although what follows is based on the best data in existence, it should be understood that, before definite conclusions can be drawn, much more detailed and extended examinations should be made.

Flood Prevention on the Lower Colorado.

It is apparent that if the flood discharge of the Colorado could be reduced materially, thereby bringing the fluctuations between low and high water within reasonable limits, the erosion of the bed and banks would be much reduced. There might still be a limited amount of bank protection required, but, if the control was such as to keep the river within banks, the cost of protective works would be greatly lessened. We have recently heard much about the control of streams by storage works; and, with reference to the Colorado, one advocate claims that storage near the head-waters will reduce flood levels so much that levees will be a thing of the past; that the flow of the stream may be entirely devoted to irrigation, thereby putting water on 5 000 000 acres of land; and also that immense quantities of electrical energy may be developed as the water is released from the places in which it is impounded. As the question of stream control by storage reservoirs has received much attention in most civilized portions of the earth, the writer has made a somewhat extended examination of the literature on that subject.

On page 406 of the "Report upon the Physics and Hydraulics of the Mississippi River," by Humphreys and Abbot, the following appears, in regard to the reservoir system:

"The plan, in theory, is admirable, and has long been a subject of discussion among European engineers. Artificial lakes for protection against floods were constructed as early as 1711 upon the upper Loire, and they have since been advocated, both for improving navigation and for restraining floods, by eminent writers, * * *."

Mr. Sellew.

From page 407 of the same publication:

"Little consideration is necessary to make it apparent that this system is not applicable to restraining the floods of all rivers. Certain topographical conditions are essential to its success. The valley must be of such a character that dams of reasonable dimensions can be constructed, which shall keep back *the identical water which otherwise would make up the flood.* It is not sufficient for this purpose, as for improving navigation, that a large volume of water may be collected by the accumulations of months. The floods of great rivers are torrents, caused by rapidly melting snows and by widely extended and heavy rains. The greater part of this water does not drain from the remote mountain sides, and issue from the distant mountain gorges. It falls in the valley itself; and the nearer to the main river, the more sudden and disastrous will be its effects; partly from the more rapid accumulation in the main stream of the contributions of the tributaries, and partly from the absence of the natural reservoir furnished by the various channels, which must be filled before a freshet originating near the sources can reach the lower part of a river. To control such floods with certainty and economy by artificial reservoirs, it is, therefore, essential that certain important tributaries which drain relatively large portions of the basin shall debouch near their mouths from narrower gorges, where dams can be constructed at reasonable cost, and where artificial lakes can be formed without injury to other interests."

From page 408:

"In order to give a more definite character to these conclusions, they will be reduced to figures by aid of the data collected respecting the great June flood of 1858, by which the merits of all these different plans of protection are to be tested.

"To have protected 'the whole delta and the borders of every stream in it, primary or tributary' against this flood, not more than 1 050 000 cubic feet per second could have been allowed to enter the head of the alluvial region. Even this quantity would have submerged much of the lower country, had not the tributaries below the Ohio been so very low that their united contributions, joined to this amount, would only have been sufficient to maintain the river at full banks. The conditions of this flood were then the most favorable possible for the reservoir system.

"During the thirty-six days in 1858 from May 25 to June 29, inclusive, the total amount of water passing the latitude of Columbus exceeded by 648 172 800 000 cubic feet that which would have resulted from a discharge per second of 1 050 000 cubic feet. Reservoirs situated above the mouth of the Ohio, and sufficient to have kept back *in a single month* fully 600 000 000 000 cubic feet of water [14 000 000 acre-ft.] would, therefore, have been essential to the security of the delta, if this system had been depended upon for restraining this flood."

From a report on the examination of reservoir sites in Wyoming and Colorado, by Hiram M. Chittenden, M. Am. Soc. C. E., Captain (now General, Retired), Corps of Engineers, U. S. A.:*

* Annual Report, Chief of Engineers, U. S. Army, for 1898, Appendix PP, p. 2845.

Mr. Sellew.

"In no other portion of her works has nature left so much to be done by the engineer to supplement her deficiencies as in the modification of the natural flow of her streams, for in no other respect are her works so ill adapted to the uses of man. The ideal stream would be one in which the flow should be uniform from one year's end to the other, or, if not uniform, varying directly with the magnitude of the uses to which it is put.

* * * * * * *

"It is not surprising, therefore, that one of the chief concerns of the engineer is the amelioration or prevention of the evils of this unfortunate arrangement of nature. Millions of dollars are annually expended to make up for the deficiency of water in seasons of drought, and like sums to prevent or alleviate the evils of excessive flow. Singularly enough, the measures generally adopted are put forward in disregard of one of the commonest rules of scientific practice. If an evil condition of things is to be corrected, the rational method of procedure is to remove the cause. In all river engineering, however, the measures adopted look only to the palliation of results, and leave the cause untouched. River channels are dredged out in low water and levees are built to protect from floods in high water. Scarcely anywhere is the effort made to prevent the occurrence of either high or low water. It would naturally follow that, if great evils result from the variable flow of streams, the primary and fundamental object of the engineer who is called upon to correct them would be to make this flow uniform. Whether or not this object is possible of realization (and if it is, by what means) is therefore one of the first questions which should be settled in any comprehensive project for the regulation of the flow of streams.

* * * * *

"* * * The only possible method by which uniformity of flow can be secured must therefore be by storing the surplus waters in seasons of flood and releasing them in seasons of drought.

"There is an additional motive for the use of reservoirs besides that of securing uniformity of flow. Over a large portion of the land area of the earth in civilized countries the climate of winter prevents any considerable use of the streams. Even if the flow were entirely uniform, that portion which takes place in the season of cold weather would mostly be lost. To derive any benefit from it, it must be stored and held over for the season of warm weather.

"These two purposes, viz., the attainment of uniformity of stream flow and the transfer of the winter supply to the summer months cover the entire argument for reservoir construction.

* * * * *

"At first thought it would seem that in storage reservoirs lies the whole solution of the river problem. To store the surplus in flood season and use it in season of drought ought apparently to strike at the root of the whole difficulty, and to render unnecessary those palliative measures which alone have hitherto received the sanction of the hydraulic engineer. Why so obvious a remedy has never yet been extensively applied will appear in the course of this report.

* * * * *

Mr. Sellew.

"It is the *cost*, not the physical difficulties, which stands in the way. It may be stated that as a general rule a sufficient amount of storage can be artificially created in the valley of any stream to rob its floods of their destructive character; but it is equally true that the benefits to be gained will not ordinarily justify the cost.*

* * * * *

"We now come to the specific question of reservoir construction in the arid region west of the one hundredth meridian, as exemplified by the reservoir sites examined in Wyoming and Colorado. Are there direct and primary motives which would justify reservoir construction in this country, apart from or in addition to those arising from their effect upon the regimen of the lower rivers? The answer must be that in no other part of the United States, nor anywhere else in the world, are there such potent and conclusive reasons, of a public as well as a private nature, for the construction of a comprehensive reservoir system as in the region here in question.†

* * * * *

"* * * in many sections the natural flow has been used as far as it is practicable to do so. The only resource left is to store that portion of the flow that runs away in nonirrigation seasons and the surplus in times of annual flood and sudden freshets and make these also available for use. Not until that is done can a stream be said to be really utilized to the fullest extent.

* * * * *

"While there are these clear and positive arguments in favor of the storage of the surplus flow of our Western streams, there are none of weight against it. It may be set down as a rule, to which there are very few exceptions, that every artificial body of water created in the West, by which the surplus water of its streams is held back, will be a positive benefit.

* * * * *

"The foregoing examination has led up to the following conclusions:

"First. A comprehensive reservoir system in the arid regions of the United States is absolutely essential to the future welfare of this portion of the national domain.

"Second. It is not possible to secure the best development of such a system except through the agency of the General Government.‡

* * * * *

"The total extent of a reservoir system in the arid regions which shall render available the entire flow of the streams will not exceed 1 161 600 000 000 cubic feet [approximately 27 000 000 acre-ft.]. If the construction of such a system were to consume a century in time, it would represent an annual storage of about 11 600 000 000 cubic feet, or 266 300 acre-feet. At $5.37 per acre-foot this would cost $1 430 031 per annum. This amount, distributed among the seventeen States and Territories of the arid region, gives an average annual expenditure in each of $84 119. The annual value of the stored water

* *Loc. cit.*, p. 2860.

† *Loc. cit.*, p. 2864.

‡ *Loc. cit.*, p. 2872.

would return the original cost and maintenance in an average period of three years."* Mr. Sellew

In a paper on "Reservoir Systems and Their Relations to Flood Protection,"† by C. O. Sherrill, Captain, Corps of Engineers, U. S. A., presented at a special meeting of the Louisiana Engineering Society on June 25th, 1912, the following appears:

"The most vital question of the moment for the people of the lower Mississippi Valley is how best to secure protection from disastrous floods, such as the one now passing, and in the proper solution of this question the sympathy and assistance of the entire country should be ready to aid. Every flood brings forth a multitude of plans, each purporting to be the only one capable of providing the necessary cure, and most of them are brought out as something entirely new, yet each one will, on examination, be found to have been carefully considered and thoroughly investigated years ago. One of these propositions, renewed recently with great energy, has been to control these floods by means of reservoirs located near the headwaters of the tributaries.

"In view of the fact that adequate reservoir systems for the control of floods in all streams would strike directly at the seat of the trouble, it seems remarkable that this method should not have been used long ago instead of the merely defensive method of elevation of overflow land or the erection of levees.

* * * * *

"* * * The question, therefore, is, Can this stream flow be made uniform; if so, how and at what cost? * * * I must answer that a reasonable degree of uniformity of flow can be secured by adequate systems of reservoirs properly located along streams where the topography is particularly adapted to such reservoirs; but as to the possibility of such control for the Mississippi below Cairo, or of the practicability of the scheme, if possible, it is hoped that the following remarks will be of some assistance in determining."

This paper concludes:

"Taking the above brief summary of the facts into consideration, I must conclude that the control of the Mississippi floods by reservoirs is impracticable of accomplishment and that the next best thing must be relied upon, namely, the levee system with bank protection, which should be completed as rapidly as possible."

The Pittsburgh Flood Commission, in a comprehensive and voluminous report covering a recent investigation, concluded that the maximum flood crests of the Ohio at Pittsburgh can be lowered approximately 8 ft. by the storage of about 1 000 000 acre-ft. of water, in seventeen reservoirs on the Allegheny and Monongahela, at a total cost of about $20 000 000.

From the foregoing it appears that the desirability and practicability of reservoir construction depend on existing conditions in the drainage area of the stream under discussion; therefore, each stream

* *Loc. cit.*, p. 2878.

† *Journal*, Association of Engineering Societies, September, 1912.

Mr. Sellew.

should be subject to careful analysis before definite recommendations can be made. As the Colorado receives almost its entire supply from melting snows at the head-waters, it would appear to be an ideal stream for reservoir control.

Existing Storage Works.—Before discussing the feasibility of reservoir control on the Colorado, attention is called to Tables 23 to 27, which show what has been accomplished by storage systems in various countries, with their extent and approximate cost.

Table 23 contains data on ten projects of the Reclamation Service having a total capacity of about 3 800 000 acre-ft., which have been constructed at an average cost of $2.65 per acre-ft.

Table 24 shows other existing reservoirs in America with an aggregate storage of 2 383 500 acre-ft., built at a total cost of $2 629 800, or an average of $1.10 per acre-ft. In this list the storage at the head-waters of the Mississippi was by utilizing natural lakes, the outlets of which were controlled by low, inexpensive dams, the average cost being only 36 cents per acre-ft. If this system be excluded, it appears that the remainder amounts to only 283 500 acre-ft., which cost about $6.65 per acre-ft. In this table there are also five sites in Wyoming and Colorado, suggested by Gen. Chittenden, aggregating 939 200 acre-ft., estimated to cost $2 709 500, or an average of about $2.90 per acre-ft. This table also shows 70 415 acre-ft. of storage in Europe and Australia, which have cost $74 per acre-ft.

Table 25 contains data on ten works in France, having a total capacity 38 593 acre-ft. and costing $3 518 900, or about $91 per acre-ft. This table also shows that 38 400 acre-ft. of storage in Austria has cost $232 per acre-ft., while in Canada reservoirs containing 3 800 000 acre-ft. are being created on numerous lakes on the Ottawa River at an estimated cost of 20 cents per acre-ft.; and further, that about 700 000 acre-ft. in South Africa have been built at an average cost of $14 per acre-ft.

Table 26 gives a list of eleven projects in Germany, volume 488 111 acre-ft., cost $22 449 874, or $46 per acre-ft.; shows that Russia has 800 000 acre-ft. on the Volga (no records of cost); and that five systems in India, aggregating 775 250 acre-ft., cost $6.77 per acre-ft. The storage of about 1 900 000 acre-ft. at the Assuan Dam, in Egypt, has cost about $10 per acre-ft.*

The completed storage (Canada being excluded as under construction) is summed up in Table 27.

Although there are many storage dams for water supply and power in various parts of the world, they are of such relatively low capacity and high unit cost that they have no place in a discussion of this kind. The foregoing data are not submitted as complete, but include all the larger systems for river control, of which records are available

* *Engineering Record*, January 11th, 1913.

Mr. Sellew.

TABLE 23.—RESERVOIRS IN THE UNITED STATES. BUILT BY THE RECLAMATION SERVICE.

Location.	Type of controlling works.	Storage, in acre-feet.	Cost.		Source of information.
			Total.	Per acre-foot.	
Arizona:					
Roosevelt Dam	Masonry.....280 ft.	1 284 000	$3 697 000	$2.90	10th Annl. Rept., U. S. R. S.
California:					
East Park	Concrete.....139 ft.	45 600	239 000	5.25	" " " "
Idaho:					
Deerflat	Earth.....40 to 70 ft.	186 000	868 800	4.65	" " " "
New Mexico:					
Carlsbad	Earth and rock.....50 ft.	47 000	220 700	4.70	" " " "
Oregon:					
Cold Springs	Earth.....98 ft.	50 000	442 600	8.85	" " " "
Klamath	Rock fill.....33 ft.	462 000 *	130 000	0.28	" " " "
South Dakota:					
Belle Fourche	Earth.....115 ft.	209 700	1 123 900	5.40	" " " "
Washington:					
Bumping Lake	Earth.....45 ft.	34 000	410 700	12.10	" " " "
Wyoming:					
Pathfinder	Masonry.....218 ft.	1 025 000 †	1 693 000	1.65	" " " "
Shoshone	Concrete.....328 ft.	456 000	1 179 300	2.60	" " " "
Totals		3 799 300	$10 005 000	$2.63	

* Lake storage.
† Drainage area, 12 000 sq. miles.

TABLE 24.—(*Continued*).

Location.	Type of controlling works.	Storage, in acre-feet.	Cost.		Source of information.
			Total.	Per acre-foot.	
MISCELLANEOUS RESERVOIRS.					
Spain:					
Villar	Masonry	13 050	$ 390 000	$29.90	Buckley.
Belgium:					
Gilleppe	Masonry	9 730	874 000	90.	"
Wales:					
Vyrnwy	Masonry	44 690	3 334 000	75.	"
Australia:					
Beetaloo	Concrete	2 945	573 300	195.	"
Totals		70 415	$5 171 300	$74.	

Mr. Sellew.

Mr. Sellew.

TABLE 24.—VARIOUS RESERVOIRS.

Location.	Type of controlling works.	Storage, in acre-feet.	Cost.		Source of information.
			Total.	Per acre-foot.	
IN THE UNITED STATES.					
Mississippi Reservoir System.	Earth and Timber Dams....	2 100 000	$ 750 000	$0.36	Pittsburgh Flood Commission.
Texas :					
Pecos River.....					
Upper Reservoir.....	Earth.....	82 640	170 000	2.06	Chief of Engineers, 1898, Ap. PP, p. 2832.
Lower Reservoir.....	Earth.....	7 000	86 000	12.29	Chittenden Report.
Colorado :					
Larimer and Weld.....	Earth.....	7 460	90 000	12.00	" " p. 2842
Cache la Poudre.....	Earth.....	5 650	125 000	22.20	" " " "
California :					
Escondido.....	Rock fill.....	3 500	110 000	31.44	" " " "
Cuyamaca.....	Earth.....	114 755	959 800	8.37	" " " "
Sweetwater.....	Masonry.....	22 500	264 000	11.70	" " " "
Bear Valley.....	Masonry.....	40 000	75 000	1.88	" " " "
Totals.....		2 383 505	$2 629 800	$1.10	
SUGGESTED RESERVOIRS—GEN. CHITTENDEN.					
Wyoming :					
Laramie.....	Masonry .....80 ft.	414 000	$1 416 000	$3.42	Chittenden Report, p. 2842.
Sweetwater.....	Masonry.....100 ft.	326 900	276 800	0.85	" " " "
Piney Creek.....	Masonry.....3 low dams	85 400	214 600	2.52	" " " "
Colorado :					
South Platte.....	Masonry.....200 ft.	67 200	540 000	8.04	" " " "
Loveland.....	Masonry.....	45 700	262 100	5.73	" " " "
Totals.....		939 200	$2 709 500	$2.90	

TABLE 25.—(*Continued*).

Location.	Type of controlling works.	Storage, in acre-feet.	Cost.		Source of information.
			Total.	Per acre-foot.	
Reservoirs in Canada.					
Ottawa River	Three projects. Low concrete dams on numerous lakes	3 800 000	$728 000	$0.20	Pittsburgh Flood Commission.
Reservoirs in South Africa.					
Cape Colony	Earth, six dams	55 422	$1 700 000	$30.80	Buckley.
	Concrete, five dams	44 274	2 800 000	63.20	"
Transvaal	Earth, one dam	95 000	684 000	7.20	"
	Concrete and weirs, four dams	505 000	4 646 000	9.20	"
Totals		699 696	$9 830 000	$14.00	

Mr. Sellew.

Mr. Sellew.

TABLE 25.—RESERVOIRS IN FRANCE, AUSTRIA, CANADA AND SOUTH AFRICA.

Location.	Type of controlling works.	Storage, in acre-feet.	Cost.		Source of information.
			Total.	Per acre-foot.	
RESERVOIRS IN FRANCE.					
Rhone River	Recommended as too expensive for reservoir control				Pittsburgh Flood Commission.
Garonne River					" " "
Loire River	Recommended for construction, but not built because of expense				" " "
Seine River	Reservoir control not applicable. Scarcity of sites				" " "
Furens Dam	Masonry, 184 ft	1 300	$318 000	$244	" " "
Ternay Dam	Masonry, 119 ft	2 100	204 372	97	" " "
Curzon	Masonry	1 297	247 600	190	Buckley.
Ban	Masonry	1 499	190 000	127	"
Pas du Riot	Masonry	1 054	256 000	243	"
Chartrain	Masonry	3 647	420 000	115	"
Lake Oredon	Earth	5 894	142 000	24	"
Mouche	Masonry	7 011	1 003 657	143	"
Liez	Earth	13 051	598 418	46	"
Wassy	Earth	1 740	138 942	80	"
Totals		38 593	$3 518 989	$91±	
RESERVOIRS IN AUSTRIA.					
Oder River	Six Reservoirs	5 100*	$1 488 000	$292	Pittsburgh Flood Commission.
Elbe and Tributaries	Four Projects proposed	13 000	2 067 000	159	" " "
Wien River	Six Projects	19 000	3 661 000	193	" " "
	One Project	1 300	1 680 000	1 292	" " "
Totals		38 400	$8 896 000	$232±	

* Drainage area, 29 sq. miles.

TABLE 26.—RESERVOIRS IN GERMANY, RUSSIA, AND INDIA.

Location.	Type of controlling works.	Storage, in acre-feet.	Drainage area, in square miles.	Cost.		Source of information.
				Total.	Per acre-foot.	
RESERVOIRS IN GERMANY.						
Wupper River:						
Bever Valley	Masonry....52.5 ft.	2 700	9	$343 200	$127	Pittsburgh Flood Commission.
Lingese Valley	Masonry....61 ft.	2 100	4	256 800	121	" " "
Ruhr River:						
12 Reservoirs	Masonry....64 to 114 ft.	33 000	71	3 480 000	105	" " "
Rur River:						
Urft Reservoir	Masonry....190 ft.	37 000	145	1 000 000	27	" " "
Weiseritz River:						
Malter Reservoir	Masonry....115 ft.	7 000	40	883 000	126	" " "
Kleingenburg Reservoir	Masonry....128 ft.	12 000	35	858 000	72	" " "
Weser River	Masonry....136 ft.	164 000	552	4 500 000	27	" " "
Oder River:						
Glatzer Neisse, Proposed	Earth....37 ft.	82 500	906	3 840 000	47	" " "
Malapane Reservoir	Earth....33 ft.	72 000	403	2 880 000	40	" " "
Tributaries in Silesia	13 Reservoirs completed or under construction..	75 000	.....	4 317 000	58	" " "
Remscheid	Masonry	811	.....	91 154	112	Buckley.
Totals	.....	488 111	.....	$22 449 874	$46	
RESERVOIRS IN RUSSIA.						
Volga Reservoirs	Low dams on numerous lakes.	800 000	.....	.....	.....	Pittsburgh Flood Commission.
RESERVOIRS IN INDIA.						
Tansa	Masonry.....	52 670	.....	$988 000	$18.76	Buckley.
Betwa	Masonry.....	36 800	.....	160 000	4.35	"
Chumbrumbaukum	Earth.....	63 780	.....	312 000	4.89	"
Bombay:						
20 tanks	Earth.....	372 000	2 500±	2 090 100	5.62	"
4 tanks	Masonry.....	250 000	450	1 700 000	6.80	"
Totals	.....	775 250	.....	$5 250 100	$6.77	

Mr. Sellew.

to the writer; and it is believed that they cover the field quite thoroughly. Mr. Sellew.

It appears that the storage in the United States (which is about 50% greater than that of the rest of the world) as listed in these tables, has cost on the average only one-ninth as much per unit of volume, while the maximum cost, that of Austria, is about 116 times the average in America. The low cost of the storage created by the Reclamation Service, when compared with that of the rest of the world, is shown in a most remarkable manner.

TABLE 27.—Summary of Completed Reservoirs.

	Acre-feet.	Cost.
Foreign.		
Egypt	1 900 000	$19 750 000
France	38 593	3 518 000
Germany	488 111	22 498 000
Austria	38 400	8 896 000
Russia—no cost recorded	.....	.....
India	775 250	5 250 000
South Africa	699 696	9 830 000
Miscellaneous foreign	70 415	5 171 000
	4 010 465	$74 913 000
$\frac{74\ 913\ 000}{4\ 010\ 465}$ = $18.70 per acre-ft.		
United States.		
Reclamation Service	3 799 300	$10 005 000
Other systems	2 383 500	2 629 800
	6 182 800	$12 634 800
$\frac{12\ 634\ 800}{6\ 182\ 800}$ = say, $2.00 per acre-ft.		

Determining Factors.—The feasibility of controlling a stream by storage will depend largely on:

(1) The cost, including the value of the area flooded, when compared with the benefits to be derived;
(2) The quantity of storage required and whether sufficient volume is available.
(3) The situation of such storage, whether it is placed so as to control properly the water-producing zone of the drainage area;
(4) The character of the water to be impounded, whether it is clear or so charged with sediment that the silting will unduly shorten the life of the works.

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These factors, as applied to the Colorado, will be considered in the foregoing order.

The Cost.—From what has already been written, it appears that, where storage exists in the arid regions, its cost is reasonable. Assume that the cheapest sites have been taken up and that future storage will cost twice as much as that already created by the Reclamation Service ($2.65 per acre-ft.), and we have $5.30 per acre-ft. This is in close agreement with Gen. Chittenden, who, as previously stated, estimated $5.35 for all storage needed in the Arid West. Of course, each site must be carefully examined, and detailed estimates must be prepared, which will no doubt vary considerably from the foregoing, but, from what has been stated, the figure given appears to be a fair and reasonable one to use in this discussion.

Volume of Storage Required.—An intelligent discussion of this phase of the matter would be greatly aided by continuous discharge measurements on the Colorado River at various points, and extending over a long period. Unfortunately, however, such measurements do not exist. The most complete records are those made at Yuma since November, 1902, by the Reclamation Service. These observations include current-meter measurements three times each week. Previous records of the flow at Yuma are based on such insufficient data—being a few scattered meter measurements from which a rating-curve was constructed—that they are of no value in this discussion. The wide variation in the quantity flowing for the same gauge height, due to the change in cross-section caused by scour, renders observations without a meter measurement practically worthless.

Table 28 shows the total discharge of the Colorado (excluding the Gila) by months, in acre-feet, the mean monthly discharge in second-feet, and also the total discharge in acre-feet per year from 1903 to 1912, inclusive. This table shows that the yearly discharge has varied from a minimum of 9 800 000 acre-ft. in 1904, to a maximum of 25 300 000 acre-ft. in 1909. The total monthly discharge has varied from a minimum of 182 500 acre-ft. in February, 1903, to a maximum of 6 397 000 acre-ft. in June, 1912.

In considering the uses of the stream for irrigation, it is proper to forecast as nearly as possible the lowest flow that is to be anticipated. In doing this, the writer has compiled Table 29, showing what may be called an "ideal minimum year", and also the monthly use of water for irrigation at Yuma. This table has been constructed by selecting from Table 28 the minimum January discharge, the minimum February discharge, etc., for all months included in the latter table. This gives a minimum year containing a total discharge of 8 315 500 acre-ft., and these discharges are shown in monthly totals on the diagram, Fig. 70.

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TABLE 28.—Discharge of Colorado River at Laguna Dam in Total Acre-Feet per Month and Mean Second-Feet.

Year.	January.		February.		March.		April.		May.		June.	
	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.
1903	185 407	3 015	182 531	3 286	367 722	5 980	788 957	13 268	1 957 151	31 830	3 116 946	52 382
1904	223 630	3 637	217 813	3 787	367 702	5 980	479 663	8 061	1 699 554	27 641	2 599 533	43 687
1905	310 680	5 053	879 894	15 843	2 087 202	33 945	1 482 419	24 913	2 293 324	37 298	4 507 351	75 748
1906	286 880	4 649	362 795	6 532	986 040	16 036	1 512 098	25 412	3 201 350	52 065	5 005 070	84 113
1907	1 256 500	20 470	980 600	17 730	1 191 000	19 410	2 028 500	34 140	2 230 000	37 900	5 640 000	94 800
1908	389 000	6 320	425 500	7 400	827 353	13 500	1 060 000	17 800	1 670 000	27 200	2 550 000	42 900
1909	543 732	8 800	597 629	10 700	827 715	13 500	1 708 984	28 687	3 310 561	53 840	6 240 488	104 873
1910	945 733	15 335	499 346	9 100	1 498 485	24 400	1 708 438	28 700	3 472 513	56 500	2 798 123	47 000
1911	541 487	8 800	742 610	13 400	1 067 700	17 400	1 213 685	20 400	2 764 960	45 000	3 818 576	64 200
1912	331 244	5 400	423 676	7 370	697 002	11 770	1 183 552	19 890	2 507 932	40 800	6 397 382	107 510

Year.	July.		August.		September.		October.		November.		December.		The Year.	
	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.
1903	2 228 065	36 735	589 166	9 581	398 736	6 701	504 869	8 211	321 271	5 399	266 598	4 336	10 907 419	15 060
1904	1 406 203	22 870	914 548	14 874	649 802	10 920	689 185	11 108	359 523	6 041	275 305	4 477	9 882 461	13 590
1905	1 859 380	30 240	744 111	12 102	383 496	6 445	482 980	7 855	440 770	7 445	571 720	9 298	16 043 327	22 180
1906	2 395 311	38 956	1 148 779	18 683	694 300	11 668	720 020	11 710	575 603	9 673	534 235	8 680	17 422 481	24 020
1907	5 930 000	96 400	2 309 600	37 600	1 286 800	21 640	778 000	12 640	629 400	10 570	458 000	7 450	24 818 400	38 230
1908	2 000 000	32 600	1 395 288	22 800	633 818	10 700	585 000	9 510	481 000	8 090	573 961	9 300	12 590 920	17 340
1909	4 875 840	79 297	2 453 986	39 910	2 807 545	47 138	860 839	14 000	561 917	9 400	517 098	8 400	25 306 334	34 880
1910	904 476	14 700	591 480	9 600	366 948	6 200	429 428	7 000	466 916	7 800	426 770	6 940	14 108 656	19 440
1911	3 048 838	49 450	1 131 983	18 400	530 388	8 900	1 726 528	28 100	705 135	11 850	465 130	7 600	17 757 030	24 460
1912	2 854 852	46 430	1 356 912	22 073	581 959	9 780	675 382	10 990	699 184	11 750	403 444	6 560	18 357 686	25 360

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For the present, the duty of water in this vicinity is assumed as 5½ acre-ft., its use being distributed throughout the year as shown in Table 29. It is hoped, and indeed expected, that this use of water will be reduced considerably as larger areas are put under cultivation and the settlers become more skilled in the application of water, but

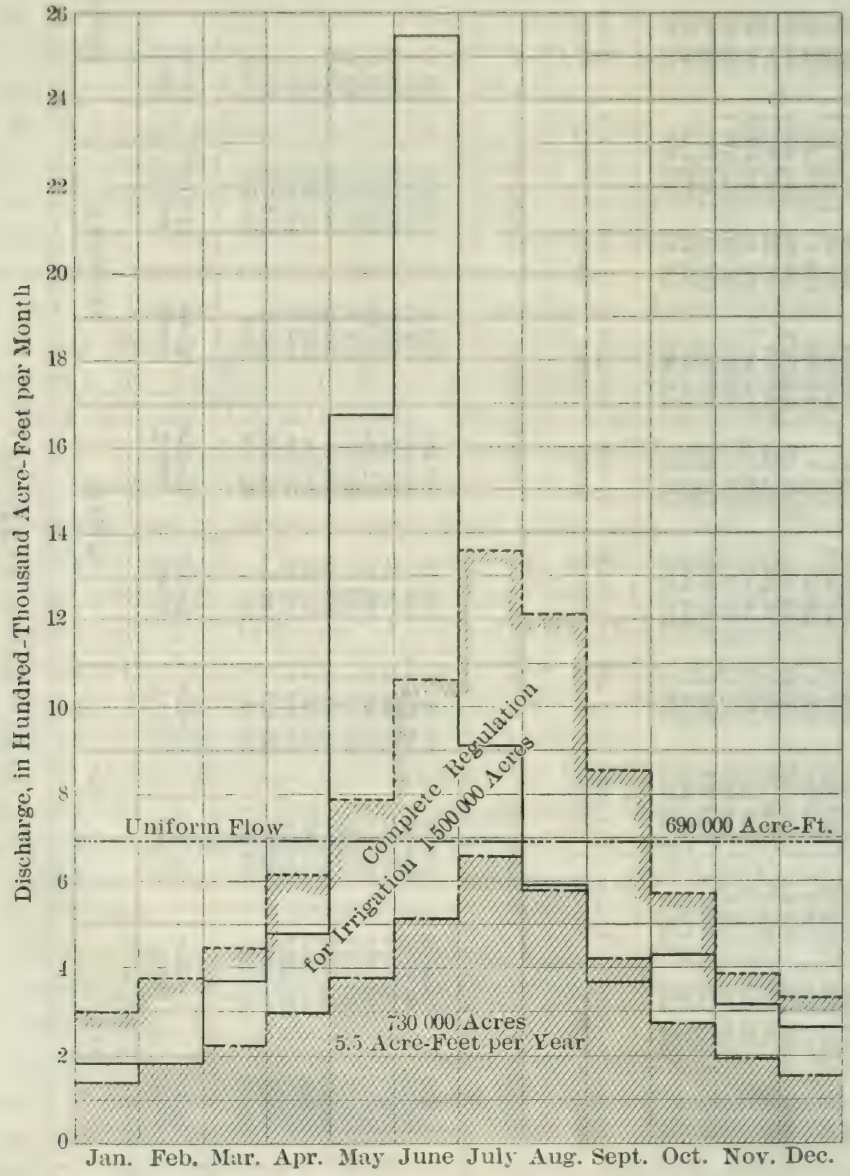

FIG. 70.

for the present this appears to be the best distribution for calculations, in view of the fact that the use now is considerably in excess of 5½ acre-ft. On Fig. 70 is also platted the use which can be made of the stream for irrigation without any regulation whatever. This was ascertained in the following manner: The use of water in February

eing about 0.25 acre-ft. and the lowest run-off in that month being 82 500 acre-ft., it was found that this quantity was sufficient for the vatering of 730 000 acres. This was assumed as the limiting month. f greater areas were placed under cultivation, there would be a short- ge in that month at a time when the new crops are usually going in. 'aking this 730 000 acres as a basis, and platting the monthly duty n the ratio shown in Table 29, it appears that the volume of water sed will be that shown by the shaded area in Fig. 70. This arrange- nent may create a slight shortage in September, but the use in that nonth is so near the minimum run-off that the danger is considered f slight importance and is neglected.

TABLE 29.—Minimum Monthly Discharge of Colorado River and Discharge in Ratio of Monthly Duty.

Month.	Minimum discharge, in acre-feet.	Monthly duty.	Percentage of total yearly duty.	Monthly discharge for year in ratio of duty.
anuary	185 407	0.20	3.6	302 801
ebruary	182 531	0.25	4.5	378 000
arch	367 702	0.30	5.4	453 500
pril	479 663	0.41	7.4	619 801
ay	1 670 000	0.52	9.4	786 100
une	2 550 000	0.70	12.8	1 058 100
uly	904 476	0.90	16.4	1 360 700
ugust	591 480	0.80	14.6	1 209 500
eptember	366 948	0.56	10.3	846 701
ctober	429 428	0.38	6.9	574 500
ovember	321 271	0.26	4.7	393 150
ecember	266 598	0.22	4.0	332 651
Totals	8 315 504	5.50	100.0	8 315 504

It is well to remark that the continuous irrigation of 730 000 acres epends on the diversion of the entire river, and is substantially all hat can be accomplished, unless the stream is regulated, even should he duty of water be increased, for it is seen that the use of water t the extremes of the year is as low as can be anticipated, and lthough a saving may be made in the summer months, it will allow n increase in the cultivated area only during those months, because he crops which can be grown continuously for twelve months in the ear are limited by the small quantity of water available during ow flow.

There is drawn on Fig. 70 a line showing the uniform flow through- ut the year, which would be at the rate of 690 000 acre-ft. per month. The establishment of such uniformity in discharge would be of value n controlling the floods and also for power purposes, but it is extremely asteful toward the irrigation interests, because this line is practically oincident with the maximum use of water in July for 730 000 acres. The reinforcement of the low flow by this method would not allow an ncrease in the area under crop in the summer. It would cover more round, however, during the winter.

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Table 29 also shows the percentages of irrigating water required per month, and on this basis the discharge of the low-water year of 8 315 500 acre-ft. has been distributed. This assumes complete regulation of the river, entirely in the interests of irrigation, and the distribution throughout the year is shown by the shaded line. Under such regulation, 1 500 000 acres (which agrees with the author's estimate) could be irrigated. This is the area that could be served with the flow of the Colorado past Yuma as observed during the years under discussion. The lower Gila is so erratic in its discharge that it is not a very dependable supply, for which reason it is omitted. To the foregoing should be added any area existing in the upper reaches already under cultivation and which it is assumed has received its supply. To regulate the river as indicated would require a storage capacity of about 2 400 000 acre-ft., which, at $5.30 per acre-ft., would cost $12 720 000. Such regulation in low-water years would also have a very beneficial effect on the flood discharge. The maximum monthly volume would be about 1 400 000 acre-ft., equivalent to approximately 50 000 acre-ft. per day, or 25 000 sec-ft. The effect of this discharge on the bed and banks would be but slight, as compared with what goes on at the present time, and would keep the lower river well within its banks.

Although this amount of storage would make the river an ideal one for irrigation during periods of ordinary low flow, it would have little beneficial effect on floods in the maximum years, as will be seen from an inspection of the diagrams on Plate LV. Here the daily discharge in acre-feet during the flood periods is shown for the years 1903 to 1912, inclusive.

When the river at Yuma rises above 125 on the local gauge, overflow occurs at various points within the limits of the Yuma Project, and levees become a necessity. If the control of the stream by storage is to regulate the discharge in such a way that levees will not be required, the volume passing when the gauge is above 125 must be stored. On the diagrams referred to, a horizontal line has been drawn representing 125 on the Yuma gauge, and the volumes of discharge above this gauge height in various years are clearly marked. They were:

In 1903	933 000	acre-ft.
1904	80 000	" "
1905	2 829 000	" "
1906	2 843 000	" "
1907	7 055 000	" "
1908	38 000	" "
1909	5 391 000	" "
1910 (the gauge did not rise above 125)		
1911	94 000	" "

PLATE LV.
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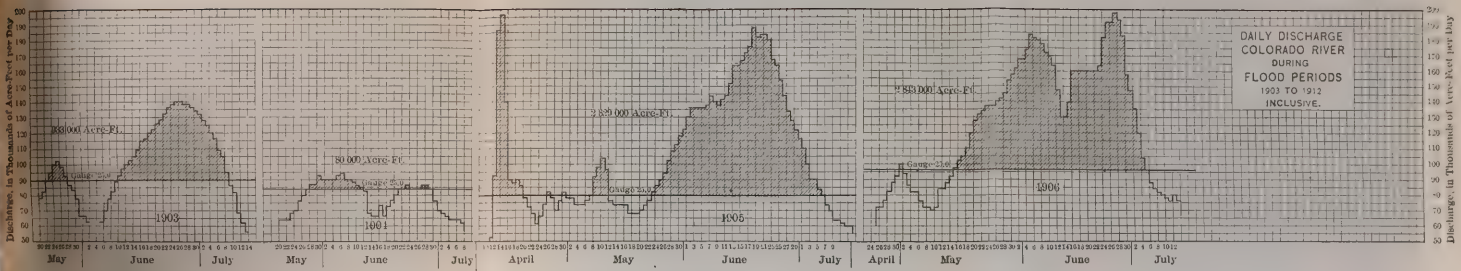

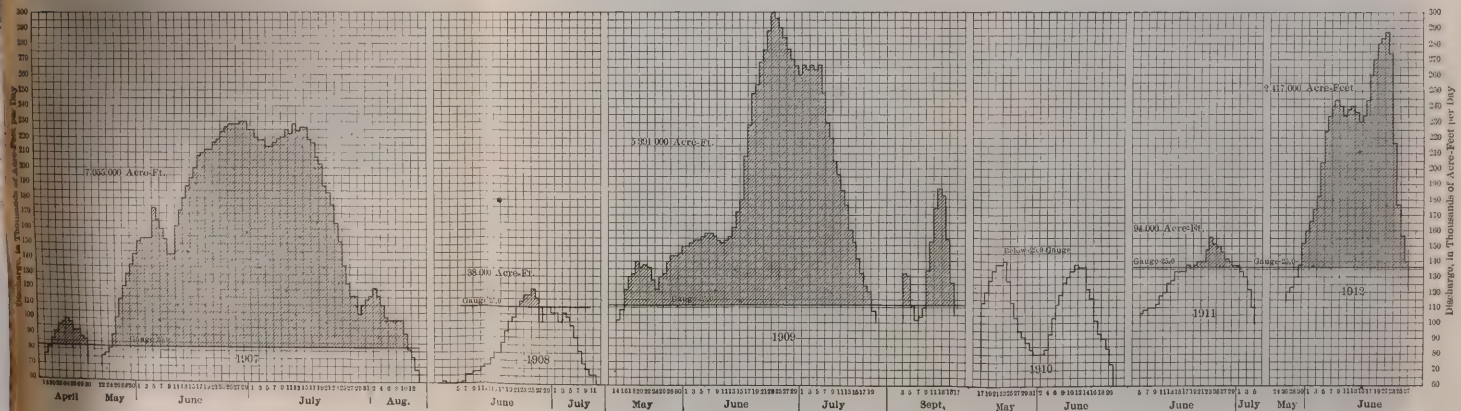

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And in the season just passed.....2 417 000 acre-ft.
A total of 21 680 000 acre-ft., or an average of 2 168 000 " "
for the ten-year period.

If storage is to be created for the maximum of these quantities, 7 055 000 acre-ft.—and this is necessary if all overflow is to be prohibited along the lower river—the cost of storage, if sufficient sites existed, would be $37 391 500, estimated at $5.30 per acre-ft. Should this be considered an excessive provision to make, and it were thought better to take what damage would come from a flood of this kind (which might be of infrequent occurrence), and storage were provided for the next largest year, which is 1909, 5 391 000 acre-ft. would be required, which would cost, on the same basis, $28 572 300. This would give a protection that would have been ample during the last 10 years, with one exception. If we take the average of the excess discharge at Yuma above the 125 gauge for the last 10 years, we get 2 168 000 acre-ft., which is about that required to regulate the low flow in the interests of irrigation. The cost of creating this amount of storage at $5.30 would be $11 490 400. With such control of the river, however, overflow would have occurred in 5 years during the last 10.

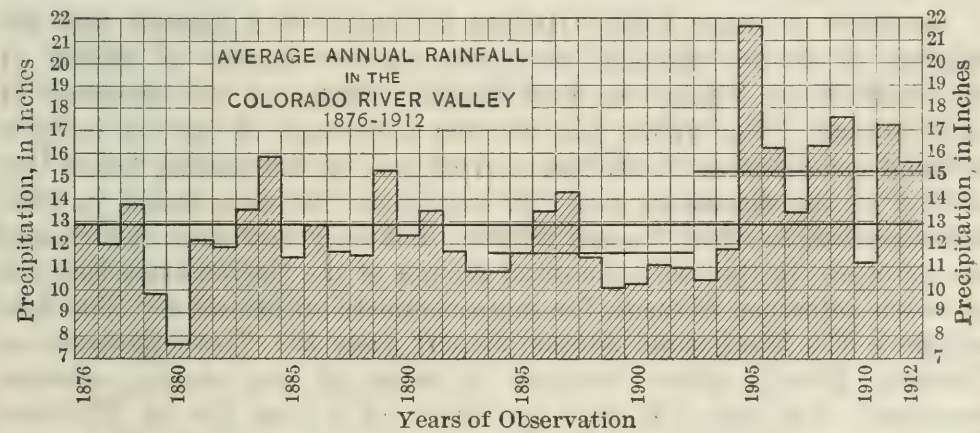

FIG. 71.

Attention is directed to Table 3, which is an exhibit of the annual discharge of the river from 1894 to 1911, inclusive, a period of 18 years. The first 9 years have an average discharge of 7 220 000 acre-ft., and the last 9 years show an average of 17 556 000 acre-ft., indicating a most remarkable, sudden, and continued increase in run-off. Such an increase might be possible with a small drainage, but, with a tributary area of more than 250 000 sq. miles, the variation appears to be altogether too much. To aid in the examination of this question, various publications of the Weather Bureau have been used in the preparation of Fig. 71. For the northern part of the water-shed, no continuous observations within the drainage area appeared to be available, there-

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fore Denver, Salt Lake City, and Cheyenne were used. These stations are probably indicative of conditions within the area under consideration. For the central and southern portions, the records of Prescott, Tucson, Yuma, Phœnix, and Flagstaff were taken. The average precipitation for the entire period is 12.92 in., shown by the heavy line. That for the period 1894 to 1902 is 11.73 in., and for 1903 to 1912 is 15.20 in. These latter amounts are indicated by the shorter and lighter lines. Thus it appears that although the run-off in the latter half of the period covered by Table 3 is about 2.4 times that of the first half, the increase in precipitation was only 1.3 times. Furthermore, it appears that the sudden increase in precipitation did not occur until 2 years after the change indicated by the table. There is no doubt that there is considerable difference in the run-off of the two periods, but it is not nearly as great as Table 3 indicates. The explanation is clear: In November, 1902, careful and systematic gaugings of the stream were inaugurated at Yuma by the Reclamation Service, and the data since collected are dependable. The data prior to that time are based on a rating-curve constructed from so few observations that the results are worthless. The first half of Table 3 should not have been presented.

Available Storage Sites.—Having indicated in a general way the volumes of storage required to attain various degrees of control of the Colorado at Yuma, the next inquiry relates to the existence of such storage. The writer has not the advantage of having made a personal examination of the upper river and its tributaries, hence this portion of the discussion must be limited to the data found in published reports to which he has access. The only authentic data found are contained in the various annual reports of the Reclamation Service, to which reference is made for more complete information. The first reference is to the Second Annual Report, page 123, "Investigations on Colorado River", where mention is made of the various schemes examined. On page 132 data are given of a dam site at Williams River, where 1 300 000 acre-ft. of storage may be created. The statement is made that investigations of a site at Bulls Head were abandoned because of poor foundation. A 100-ft. dam at this point would provide storage for 845 000 acre-ft. The lower end of Pyramid Canyon was examined. Surface conditions appeared to be favorable for a high dam, but no suitable foundation was found. If it were possible to build dams at these points, the scheme would be impractical because the reservoirs would soon be filled by the deposition of sediment, which will appear from data given later. Reference is made to this difficulty in the Second Annual Report, page 155, where we may read:

"In connection with this question of the study of reservoirs which silt up, it seems most necessary that a thorough investigation be made of the reservoir possibilities upon the head-waters of the Colorado,

above the points where the streams carrying large quantities of silt enter, so that permanent reservoirs may be established in anticipation of the time when all the irrigable lands along the Colorado will be under ditch." Mr. Sellew.

Thus the Second Annual Report shows no available storage sites for the permanent control of the river.

The Third Annual Report, page 70, has the following:

"Colorado River Storage Projects.—As it has been thought necessary to store water in the Rocky Mountain region for the regulation of the flow of the Colorado River, with a view to the reclamation of about 600 000 acres of land along the lower Colorado in Arizona and California, certain reservoir sites have been withdrawn in western Colorado, two of these being on Grand River, in Grand County, and two on Yampa River, in Routt County. Preliminary examinations have been made of all these sites, and some of them have been surveyed."

On page 217, the Kremmling site is credited with a capacity of 1 500 000 acre-ft. At Grand Lake about 23 000 acres are reported as withdrawn, but no capacity is given. Two sites on the Yampa were located and 100 000 acres withdrawn, but no capacity is given, as the data were not complete.

In the Fourth Annual Report, page 121, it is stated that at the Kremmling site more than 1 000 000 acre-ft. may be stored by a dam 180 ft. high, and that a 230-ft. dam would impound 2 200 000 acre-ft.

On page 124 reference is made to further examinations on Grand River above the Kremmling site, where it is stated that: "Few reservoir and dam sites worthy of investigation were encountered during the reconnaissance."

Reference is made to:

Windy Gap site—capacity	100 000	acre-ft.
Lehman site—capacity	24 000	" "
Grand Lake—capacity	140 000	" "
	264 000	acre-ft.

The first site is crossed by a railroad; would require a large dam; and 75% of the area flooded would be ranches already under cultivation. The second site would cover valuable land under cultivation, and is of slight consequence in size. Regarding Grand Lake, the report says, on page 127:

"Considered as a site for storing irrigation water for use along the lower Grand and Colorado rivers, Grand Lake, as will be readily seen from the foregoing, would be of little value."

It appears, then, that this report shows no storage sites of value except the Kremmling, with a possible capacity of 2 200 000 acre-ft.

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The Fifth Annual Report states, on page 114, that the field work in connection with reservoir sites on the Colorado was brought to a close late in 1905, and no additional examinations have been made since those described in the Fourth Annual Report. The Sixth Annual Report gives no additional information regarding this matter. On page 57 of the Seventh Annual Report reference is made to investigations for storage reservoirs along the Green River and its tributaries, from which the following is obtained:

Flaming Gorge site—capacity.....	350 000	acre-ft.
Island Park site—capacity.....	150 000	" "
Browns Park site—capacity.....	2 500 000	" "
	3 000 000	acre-ft.

No additional information is contained in the Eighth, Ninth, or Tenth Annual Reports.

Taking from the foregoing those sites which appear to be available for storage on the tributaries of the Colorado, we have:

	Capacity in acre-feet.
Grand River: Kremmling site.....	2 200 000
Green River: Flaming Gorge site.....	350 000
Island Park site.....	150 000
Browns Park site.....	2 500 000
Total.....	5 200 000

Railroad interests have secured control of the Kremmling site, therefore its future development may prevent the creation of reservoirs of any magnitude. Such a result would remove the only known reservoir capable of controlling the Grand River, and would reduce the known available storage for uses on the Lower Colorado to 3 000 000 acre-ft., all confined to the Green. This is but little in excess of that required for minimum-year control in the interests of irrigation, and furnishes little relief in the years of maximum flow, when, as previously shown, 7 000 000 acre-ft. must be stored in order to prevent overflow, unless dependence is placed on levees. The foregoing data are not sufficient to determine with accuracy the feasibility of reservoir control on the Colorado River. Before conclusions can be properly drawn, investigation should show the total available storage, its geographical distribution, and whether it is placed so that it will control the run-off from the various parts of the catchment area. Should sufficient storage exist to save the surplus of the maximum years for use in years of low flow, the irrigable area along the Colorado could be increased correspondingly.

The Quantity of Silt.—As previously stated, reservoirs created on the lower portion of the stream would very soon fill with deposits, as

the Lower Colorado is a notorious silt carrier. Professor R. H. Forbes* says, regarding Colorado silt: Mr. Sellew.

"Beginning with January, the regular low winter water averages about 62 parts of sediment in 100 000 parts of water during January, February, and March. These sediments are probably chiefly the result of erosion upon the upper river channel, rather than surface sweepings into the drainage. During April, as the river rises with the melting snows of its highest water-shed, and its erosive power increases, the sediment rises to an average of 112 parts; during the highest waters of May and June, to an average of 374 parts; decreasing with the lowering waters of July and August to an average of 122 parts. The summer rain-storms of September, October, and November, partly within Arizona, bring the silt to its maximum (2 072 parts, Oct. 8-13), after which the quantity sinks to the normal amount for winter waters, averaging 151 parts in 100 000 during the two months ending Jan. 24, 1901.

"* * * The average acre-foot of Colorado river water for the year of observation carried 7 291 pounds of silt, varying from 1 182 to 56 410 pounds as shown by 6-day composite samples.

"On the basis of the profile constructed from available data for the volume of flow of the Colorado, and of the year's silt determinations made in this laboratory, it is estimated, conservatively, that the river during 1900 brought down about 61 000 000 tons of sedimentary material, which, condensed to the form of solid rock, is enough to cover 26.4 square miles one foot deep; or, to make 53 square miles of dry alluvial soil one foot deep; or, to make about 164 square miles of recently settled, submerged mud one foot deep, reckoning the whole amount of mud for the year to average 6.2 times the bulk of the solid sediment."

From these extracts it appears that in 1900 the volume of sediment varied from 0.062% in January, February, and March, to 0.374% in May and June, and that following desert rains it was as high as 2.07 per cent. An area of 164 sq. miles of mud 1 ft. deep equals 105 000 acre-ft. Since May, 1909, the Reclamation Service has observed the quantity of sediment passing Yuma, the results being as shown in Table 30.

TABLE 30.—Quantity of Sediment in Colorado River.

Year.	Total discharge, in acre-feet.	Number of silt observations.	Average percentage of silt, by weight.	Acre-feet of submerged mud annually.
1909.....	25 967 577	8	0.65	321 010
1910.....	14 332 692	22	0.50	136 660
1911.....	17 839 245	52	1.15	391 115
1912.....	18 357 686	64	0.756	265 734
Total..	.....	.....	.....	1 114 519

Bulletin No. 44, Arizona Experiment Station, pp. 198 and 200.

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The observations of 1909, although few in number, covered the period from May 26th to October 7th, and should give a fair average. Those of 1910 extended from April 19th to December 31st. In 1911 they extended from January 11th to December 28th, and in 1912 from January 4th to November 30th, the discharge being estimated to December 31st. As shown in Table 30, the total quantity of sediment passing Yuma during the last 4 years, when measured as submerged mud, has exceeded 1 000 000 acre-ft. This proves conclusively that reservoirs for the permanent control of this stream must be placed on the head-waters where clear water exists.

The author shows in Table 2 an approximate storage of about 10 000 000 acre-ft., and although he qualifies the table by stating that it is "theoretically possible" and would have to be reduced for "commercially feasible developments," the table is so at variance with facts as to be misleading in its present form, and likely to lead uninformed persons to erroneous conclusions. Storage on the Gila is out of the question, as this stream carries more silt during its flowing season than the Colorado. Little Colorado, Bill Williams Fork, and San Juan must be eliminated for the same reason. The Kremmling site is in the hands of interests foreign to irrigation or storage development. The only site of value contained in Table 2 is Browns Park on the Green, which has a capacity of about 2 200 000 acre-ft., as previously indicated. This analysis reduces the author's "theoretically possible" 10 150 000 acre-ft. to 2 200 000; reservoirs which will silt up in a few years or are controlled so that they cannot be developed, are not in the "possible" class.

There are some statements which the writer cannot reconcile with the facts as they have come under his notice. The area of the Yuma Project is some 130 000 acres, instead of 90 000. Table 4, showing mean monthly discharge from 1894 to 1911, is valueless, for the results from 1894 to 1903 are based on erroneous data, as previously shown.

Rise of Bed.

The data in regard to the rise of bed at Yuma are based on observations made largely by the Reclamation Service, but are not considered by the writer as sufficiently definite to justify the conclusions. Although the advance of the delta into the Gulf will tend to raise gauge heights at points above, certainly many more observations are necessary before definite relations can be established between the rise on the Yuma gauge and the memory of a steamboat captain.

There is at least one other cause for variation in gauge height at different points on the stream, which is entirely dissociated from any advance into the Gulf. There are two points on the lower river in this vicinity which may be regarded as definitely fixed. They are Laguna Dam and the conglomerate hills at Yuma between which the river

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PLATE LVI.
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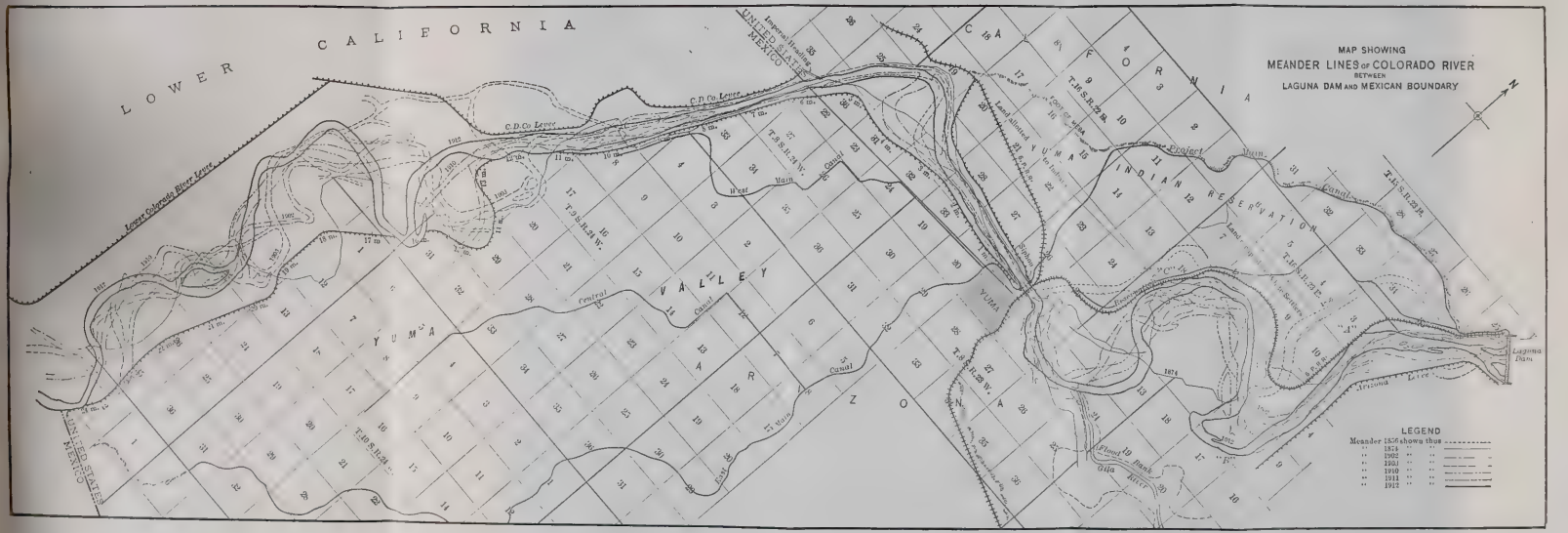

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passes. The floods of 1909 and 1912 carried substantially the same quantities at their crests, 150 000 sec-ft., but the difference in the elevation of water surfaces between the Yuma gauge and Laguna Dam was 4 ft. greater in 1912 than in 1909. The casual observer might think that this indicated a rise in the bed of the stream. It is found, however, that the slope per mile was substantially the same, being 1 ft. in 1909 and 1.04 ft. in 1912. The rise is not due to any silting up of the channel, but is plainly caused by the meanders of the river. The records show that the distance from Laguna Dam to Yuma has varied as follows:

In	1856	13	miles.
	1895	11¾	"
	1902	14	"
	1909	13¾	"
	1911	15¼	"
	1912	17	"

Some of these various meanders are indicated on Plate LVI. The river in 1912 between the points mentioned was 3¼ miles longer than in 1909, and this accounts for practically all the increased difference of elevation in the water at the two points. The same phenomenon is true on the lower river, and, assuming a given elevation at the Mexican boundary, the increase in length of the stream due to meanders will give an increased height on the Yuma gauge, whether the river is advancing into the Gulf or not.

The author's description of the works of the Yuma Project and remarks relative thereto, indicate unfamiliarity with the subject. A very excellent account of the work at Laguna Dam may be obtained from articles* by the Resident Engineer, Edwin D. Vincent, M. Am. Soc. C. E. The Colorado Siphon has been described by the writer.† These cover the main features of the work. Levees and canals are so common, and those at Yuma present no unusual conditions, that description appears to be superfluous.

The author's statement, that a rock fill such as was used for closing the crevasses which he discusses could have been used in place of Laguna Dam, is open to serious doubt. What he built has been used as a levee, which purpose it answers admirably. How it would act as an overflow weir, carrying 150 000 sec-ft., even if covered with concrete, is problematical. In addition to a concrete top, crest and footwalls appear to be necessary to prevent the top from being undermined, and, if these are provided, the resulting structure will not differ materially from Laguna Dam either in design or cost.

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* *Engineering News*, February 27th, 1908, and June 10th, 1909.

† *Engineering News*, August 29th, 1912.

LEVEES.

Mr. Sellew. The statement that the flood of December 7th, 1906, caused levee failures on the Yuma Project due to the absence of muck-ditches, is not correct. In October, 1906, the writer took charge of the Yuma Project, and his first act was to order muck-ditches under all new levees. The slight damage done here by the flood of December 7th, was caused by burrowing animals and the overtopping of work which was incomplete.

Regarding the advantages and disadvantages of land-side and river-side borrow-pits, the writer has fully expressed himself,* so that it is not necessary to comment further. Particularly is this so when the author states that he "realized that this was not in accordance with the usual practice." Based on a personal examination made a few hours after the break of December 5th, 1906, the writer hazards the opinion that the damage caused by a head of 15 in. against the levee was due primarily to the presence of land-side borrow-pits, although small quantities of water came under the levee itself in many places due to the absence of a muck-ditch and the improper grubbing of the foundation.

Regarding the loan of a grading camp by the Reclamation Service, which outfit the author declares did work at excessive cost, the following is submitted:

Immediately following the break, in December, 1906, the writer discussed with Mr. Cory the feasibility of assembling a sufficient force to construct levees required before the coming of the annual freshet. At that time it was not known how much levee work would be done, as the surveys were incomplete. Consequently, no contracts were let, and the size of the outfit which could be immediately available was not known. Because of various interests at stake, the writer suggested that, to assist in overcoming the difficulty, one camp of Government stock then operating on the Arizona levees be transferred for the use of the California Development Company. Mr. Cory welcomed this suggestion; authority was secured from the Department; and the camp was immediately transferred. It was supposed when the transfer was made that the camp was to work for the California Development Company and not for a contractor. However, shortly after the move was made, the superintendent of the camp informed the writer that he was working under the direction of a contractor, and it was next to impossible for him to obtain the proper supplies to keep his camp up to efficiency. A visit to the camp showed the facts to be substantially as stated, and a personal appeal was made to Mr. Cory to remedy the matter. This was remedied, in a measure, but conditions continued to be such that, as soon as it was evident that the camp was

* *Engineering News*, February 15th and April 11th, 1912.

not necessary to secure the completion of the levees in advance of the annual flood, the men and equipment were brought back to the United States. Mr. Sellew.

The camp that was loaned the C. D. Co. had been at work for considerable time prior, for the Reclamation Service, and continued to be thus employed for several years thereafter. It was one of the most efficient camps which the Service has had on levee construction. There have been completed on the Yuma Project about 50 miles of levee, aggregating more than 2 600 000 cu. yd. of excavation, the accomplishment of which compares most favorably in every way with any levee work which has come under the writer's notice. Criticism of Government work seems to be a general pastime of people who are not responsible for it, and certainly if a contractor desired to place a Government camp in an unfavorable light, Mr. Cory granted him a most excellent opportunity to do so.

Under the head of "Criticism of Levee Work Done" the author shows that after a Consulting Board from the Reclamation Service had advised on the methods of levee construction, their recommendations were ignored and a second Board was necessary before the recommendation as to the location of borrow-pits was complied with. The author states that the use of the checker-board system of borrow-pits increased the cost of the work about $6\frac{3}{4}$ cents per cu. yd. Experience on the Yuma Project has shown that an increase of 2 cents per cu. yd. is ample to cover this kind of borrow-pit, and the writer believes that the protection thus afforded fully justifies the additional expense.

International Negotiations.

International agreements are necessary to a comprehensive plan for the control of the Lower Colorado. Such agreements must be preceded by diplomatic action, and while such negotiations may be in progress, discussion of this phase of the matter appears to be inopportune.

The Abejas Diversion.

The author is correct in the statement that this diversion has had little effect on river levels at the entrance of the Imperial Canal. The observations show that it has had no effect whatever. The writer has little first-hand knowledge of the work at the Abejas, carried out under the direction of John A. Ockerson, Past-President, Am. Soc. C. E., so does not feel qualified to discuss it. He wishes to state, however, that the delay in starting the work, due to causes over which Mr. Ockerson had no control, which exposed the operations to the menace of the flashy winter floods, had much to do with the progress. It is believed that the Ockerson plan to levee the river to the range of tidal influence is the correct one; until this is done and the levees are made

Mr. Sellew. secure against the attack of the current, the Lower Colorado cannot be regarded as under control.

The Real Problem.

The author concludes:

"Because of the various successful and unsuccessful work done in the region, the engineering features of irrigation and river control along the Lower Colorado are now understood, and engineering construction methods are thoroughly developed.

* * * * * *

"The Colorado River Delta now presents no unusual, unsolved engineering difficulties; its problems are chiefly matters of statecraft, in both river control and irrigation."

After more than 6 years of service, during which time about 45 miles of the lower river has been in his charge, the writer wishes to state that, in his judgment, the real problem of this river is not generally understood. This problem, for the solution of which the author offers no aid, is bank protection. Construction methods for its accomplishment are not developed, and it cannot be attained by statecraft, however high the talent engaged. Relief from this menace will come only from hard work and a liberal expenditure of money on the immediate banks. When levees are undermined while the stream is below bank-full stage, it is clear that many years must elapse before a river of this magnitude will be controlled by reservoirs in such a way that caving banks will be eliminated. A plan by which these banks may be made stable will now be outlined, but, before proceeding with it, a short sketch of the flood of 1912 may be of interest.

Lower Colorado Flood of 1912.

The annual freshet of 1912 first made its appearance on the Yuma gauge on March 12th, the reading then being 117.8 ft. and the discharge 12 800 sec-ft. It continued to rise until June 22d, when the gauge was 129.1 and the discharge was nearly 150 000 sec-ft. During this rise the erosive action of the current was causing radical changes in the bed and banks of the stream, as may be seen by Fig. 72, showing conditions at the gauging station. It is there seen that at low water the width of the river was 420 ft., its average depth about 3½ ft., maximum depth 8 ft., and the area of the flowing prism 1 420 sq. ft. At the crest these dimensions had become, width 600 ft., average depth 35 ft., maximum depth 53 ft., area 20 740 sq. ft., the discharge having increased 43 times and the area about 15 times. This increase in area, of 19 320 sq. ft., is made up of 7 440 sq. ft. caused by the rise on the gauge and 11 880 sq. ft. (60%) which was the increase in section due to scour. These modifications in cross-section were probably maximum at the gauging station, where the width of the stream is restricted; but

similar changes, although less in magnitude, are characteristic of the lower river. Mr. Sellew.

During the rise, extensive meanderings were in progress at various points, the first attack which required defense being between 10½ and 12 miles below Yuma, on the Arizona side, where the levee had been cut out by the river in 1909. During 1909 and 1910 efforts were made to hold this portion of the bank with fascines and other light brush protections, but such work was soon carried out, and resort was then had to permeable dikes of piles and brush. About seventeen permeable dikes have been built, beginning in May, 1911. Some of these dikes are shown on Figs. 73 and 74. At this point the Steamer *Searchlight,* with a pile-driver barge, worked throughout the flood of 1912, building dikes and defending the bank between them with

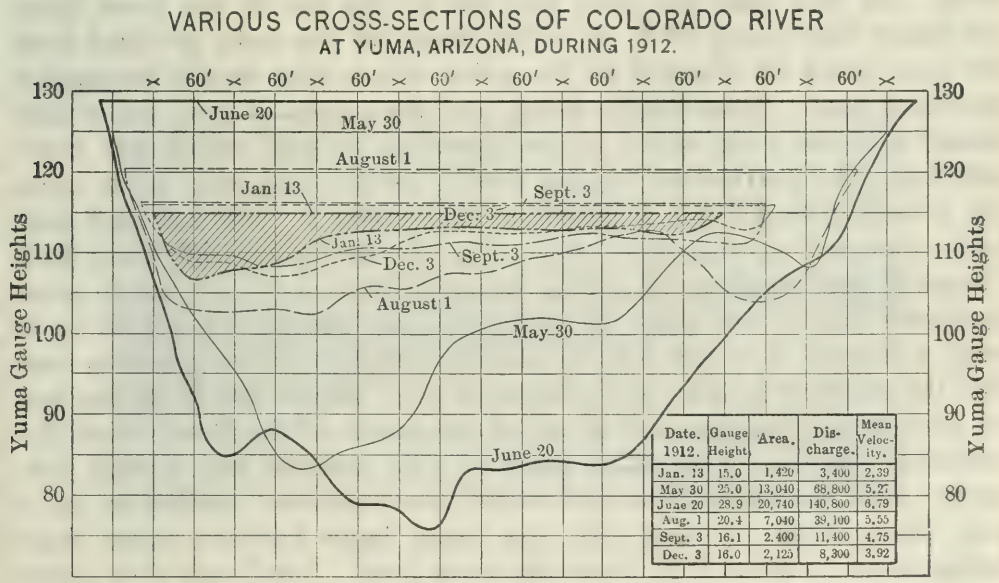
VARIOUS CROSS-SECTIONS OF COLORADO RIVER
AT YUMA, ARIZONA, DURING 1912.
60′ 60′ 60′ 60′ 60′ 60′ 60′ 60′ 60′
130
120
110
100
90
80
Yuma Gauge Heights
June 20
May 30
August 1
Jan. 13
Sept. 3
Dec. 3
Jan. 13
Dec. 3
Sept. 3
August 1
May 30
June 20
Date. 1912. | Gauge Height. | Area. | Discharge. | Mean Velocity.
Jan. 13 | 15.0 | 1,420 | 3,400 | 2.39
May 30 | 25.0 | 13,040 | 68,800 | 5.27
June 20 | 28.9 | 20,740 | 140,800 | 6.79
Aug. 1 | 20.4 | 7,040 | 39,100 | 5.55
Sept. 3 | 16.1 | 2,400 | 11,400 | 4.75
Dec. 3 | 16.0 | 2,125 | 8,300 | 3.92

FIG. 72.

fascines. A temporary levee also was thrown up when the river threatened overflow. The efforts here were successful, the dikes holding well and preventing further recession of the bank line, and the temporary levee removed all danger from overflow.

The first attack on the Reservation levee was at the point, *C,* on Plate LVI. At the beginning of the rise, the bank at this point was some 800 ft. from the levee, but on June 15th bank caving set in, and on June 17th, when the river was within 150 ft., defensive operations were commenced. As all our floating plant was engaged below Yuma, the fight here had to be made from the shore. When the stream went overbank the bottoms were flooded to within 150 ft. of the levee, making defensive operations impractical until the caving reached this point. Fascine mats weighted with sand sacks were hung over the face of

Mr. Sellew.

the bank, but, as the water was more than 30 ft. deep, such defenses would not reach the bottom, and, although our work delayed the advance of the stream, it was slowly undermined, demonstrating that a strong revetment would be necessary to hold the river in check. Conditions along the front at this point are shown on Fig. 75. Fortunately, the Reservation levee furnishes the location for the Potholes Branch of the Southern Pacific Railroad, therefore, rock, which was the only material that would save the day, could be delivered at the site of the works and dumped directly into place. Rocks weighing from 1 to 6 tons were ordered from the Declez granite quarries some 200 miles west of Yuma on the main line of the Southern Pacific. The railroad expedited the shipment, and within 36 hours after the order was placed the first train load, consisting of fifteen cars, was being thrown into the river. At this time the river had cut to the toe of the levee slope, the water there being 32 ft. in depth, therefore the rocks as rolled from the cars could be directed to the point where they would accomplish the most good. Considerable brush and thousands of sand sacks were mixed with the rock, also a limited quantity of spoil which was available from the quarries at Laguna Dam. As the flood came up to crest, the attack extended for a length of about 900 ft. Several bad sand boils appeared at the inner toe. Fig. 76 shows one of these boils "hooped" with sand sacks. This menance became so serious in some instances that at one time the destruction of the levee appeared to be only a matter of a few hours, and arrangements were made to confine the overflow in case the levee went out. About 500 ft. to the rear of, and parallel to, the levee is one of the canals of the Reservation distribution system. The outer bank of this channel was hastily connected with the levee above and below the threatened portion by sandbag dams (Figs. 77 and 78), the canal bank between these dams being raised to about 1 ft. above the river level. Sand bags were necessary as the ground, because of the seepage, was under several inches of water, and stock could not be worked upon it. These dams and the canal bank would have confined the overflow to this area had the levee gone out. It was also intended to flood this enclosed area to a depth of 2 ft. or more to check the sand boils, water for this purpose being supplied from the canals, but the boils were checked by a liberal use of sacks along the front before such action was necessary. Figs. 79 and 80 show the character of the rock used for protection, and Fig. 81 is a view of the Steamer *Searchlight* engaged in dragging the larger rock from the cars with her steam capstan, the boat having been temporarily transferred for this purpose from the work below.

The flood crested at about 1 A. M. on June 23d, and then fell rapidly. By June 25th there was no further danger of overflow, but dumping the spoil, in order to blanket the outer slope and reinforce the levee, continued until August 18th.

FIG. 73.—PERMEABLE DIKES.

FIG. 74.—PERMEABLE DIKES.

FIG. 75.—CONDITIONS ALONG THE FRONT.

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Mr. Sellew.

While this attack was in progress, another was initiated by the river at the point, *A*, on Plate LVI. Here the caving banks came to the levee toe with a water depth of 30 ft., and the defense was practically a repetition of the one just described.

On June 25th, the river having fallen below the overflow line, shipments of Declez rock were stopped; a steam shovel was placed in commission at Laguna Dam, and a work train and crew were obtained from the Southern Pacific Company, which, with assistance from our dinky engines, continued to dump rock and spoil at the points, *A* and *C*, until August 18th, when 18 000 cu. yd., including 1 300 cu. yd. of Declez rock, had been placed. On this date the threatened levees were safe from further attack, and the work was stopped. At the present time the river lies against the rock slope of the levee at *A* for 1 500 ft. and at *C* for 1 000 ft. Should these conditions remain until the next flood, further rock revetment will probably be necessary immediately above and below each of these points, therefore steam shovels, cars, and engines are being placed in condition to do this work if required.

While the attack on the Reservation levees was going on, rapid bank caving was also in progress on the Arizona side below Yuma. The work at the 12-mile post has already been briefly described. At the 16-mile post the river was 600 ft. from the levee on May 25th. On June 1st it had moved out to 1 300 ft. On June 10th it had returned to within 500 ft., on June 20th about 300 ft., and on June 25th it cut through the levee. When the caving had approached within 300 ft. of the levee, provisions were made to protect the work. There was no floating plant available for this protection, and, had there been, no effective use of it would have been possible, as the depth of water and velocity of the current made the construction of spur dikes impracticable. As there was no railroad on the levee, rock could not be obtained, as the roads to the nearest quarry, 12 miles away, were impassable for heavy loads. The only defense that could be offered was to protect the bank by fascine mats, there being plenty of suitable willow brush near at hand. The force employed, consisting mainly of Cocopah Indians, was rather indifferent labor, but by diligent efforts the rate of caving was delayed so much that, when it reached the levee on June 25th, the river had fallen below the natural bank level and no overflow occurred.

Shortly after the cutting out of the levee the river swung away for several hundred feet, and later came back for a portion of this distance, the present position of the river being shown by the meander line on Plate LVI.

About July 1st, the river being well within its banks, the patrol which had been maintained during the flood was removed. On this date no serious cutting appeared at any point between Yuma and the International Boundary Line, on the American side of the stream.

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On August 10th it was found that the river had cut through the levee at the 24-mile point, but the bank here being above the river level, no overflow had occurred. On June 20th, or a few days prior to the crest of the flood, the bank at this point was 1 300 ft. away from the river, and there appeared to have been no material change on July 1st, when the patrol was taken off; thus it appears that the caving here must have been extremely rapid.

All work of bank protection was continuous, day and night, while the danger of overflow existed. During this period the temperature in the day averaged 106° and at night about 73 degrees. Because of the extreme heat, the work was very hard on the men, and at times our efforts were greatly hampered by lack of labor. All the Indians in the vicinity were pressed into service. Wages were temporarily increased to attract idle men in the Town of Yuma, and on the Reservation the settlers whose lands were threatened turned out practically to a man and fought valiantly in the defense of their homes.

Future Bank Protection.

The work of bank protection should be directed:

(1) Toward holding existing levees and regaining lost ground, in order that work which has been destroyed may be rebuilt near its original position, thus maintaining as nearly as possible the irrigable area;

(2) To cause the river to occupy a position about midway between the levees in order to prevent, if possible, the erosion of one season from reaching the embankments; or the river slopes of the levee should be made impregnable to river attack.

It is also desirable, in order to prevent meandering—which, as shown previously, may result in a rise in plane—to confine the stream to the shortest route consistent with a fall of about 1 ft. per mile.

Regarding the recovery of lost ground, the writer is aware that the building of levees on their old locations has not been generally practiced. The following, relating to the Mississippi, is from *Occasional Papers No. 41,* Engineer School, U. S. A., page 278:

"*Location.*—The two objects of levees, protection of land and concentration of flood discharge, will be most efficiently obtained by building the levees as close as possible to the river bank, and, as owing to the general slope of the bottom land, the banks are usually higher than the land farther back, their location close to the river satisfies generally the two conditions of maximum usefulness and minimum cost, and if the river were straight and its banks stable no better location could be sought; but the river is not straight and the banks cave rapidly, and a location close to the river therefore has some disadvantages. On some of the long points of the river a levee following the bank will have, compared with a line across the point, a length

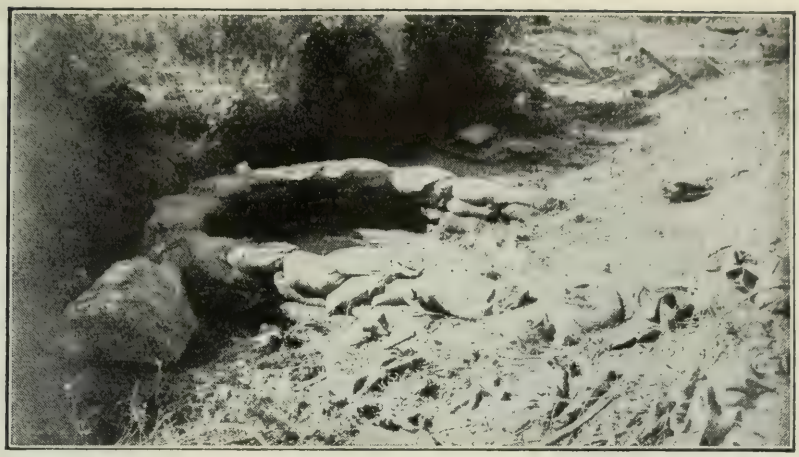

FIG. 76.—SAND "BOILS" HOOPED WITH SAND SACKS.

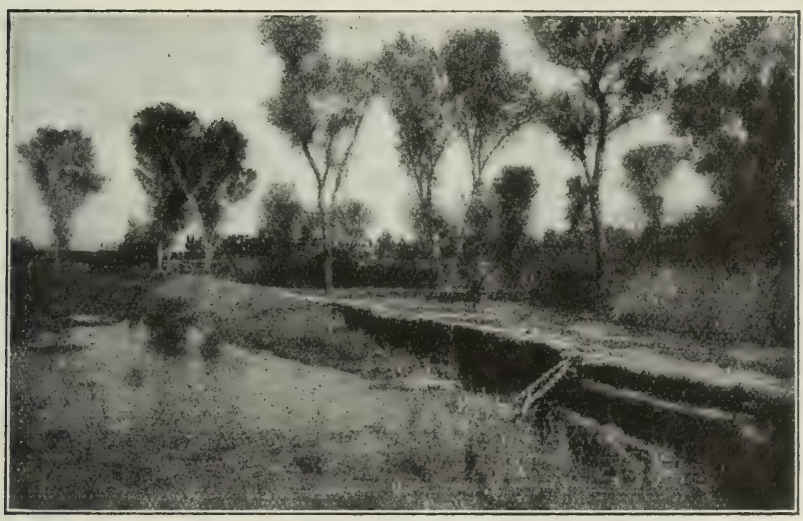

FIG. 77.—SAND-BAG DAM.

FIG. 78.—SAND-BAG DAM.

not justified by the small additional amount of land protected. In a rapidly caving bend a levee close to the river bank must in a short time be destroyed. Complications are also frequently introduced by the existence of the old river lakes caused by cut-offs. The short and direct line would pass between the lake and the river, but such a location requires usually a high and large levee and involves danger from the nature of the soil recently deposited in the old river bed and the probable existence of permeable sub-strata liable to be washed out by hydraulic pressure during floods. On the other hand, the line around the lake, though safe, is apt to be of extreme length. Mr. Sellew.

"The question of levee location is, therefore, not a simple matter and should receive careful consideration, but such consideration has, unfortunately, not been always given to it. Most of the existing levees have grown up gradually by the incorporation in the main system and successive enlargement of old levees built by private parties for their own protection, and the location of these old levees, having been made without any reference to a general system, is, of course, in many cases, faulty. Even where the levee has been built entirely anew the location has been most usually fixed by the local boards, who are, of course, subject to local influences, and many levees have thus been located too close to a caving bank, in order to include special farms.

"But even when a new levee is to be located with reference only to its general use, the question is not an easy one. If the banks of the river cave with any regularity, the cost of a new levee and its life in any location could be balanced against the value of the additional protection obtained by this location. But the caving does not go on with any sort of regularity; some places that may not have caved for years will suddenly begin to cave rapidly, and at other places where caving has long been going on the caving will sometimes somewhat suddenly cease. In placing levees near caving banks, therefore, a careful study must be made of the locality and of the river for some miles above, and, in general, it is considered a good location for the levee if this study indicates a probable life of about twenty years."

Although 20 years was formerly considered economic practice on the Mississippi, recent developments have modified it, as shown by the following:*

"It is the judgment of the Commission that these revetment works and some others which will be imperative within a short time are required for the preservation of the levee system. Without them a permanently completed system will be impossible. The increasing height and cost of the levees and the rapid development of the country protected by them greatly augment the disaster of crevasses. A break in an embankment 5 or 6 feet high resulting in the thin overflow of a sparsely populated country is a small calamity compared with a break in a levee 20 feet high, discharging over a thickly populated country threaded with railroads and filled with towns and factories. Owing to the increased height now necessary to control the floods the expense of replacing an undermined levee by a new line going far around a lake or impassable swamp is enormous. The point of development

* Report of Mississippi River Commission, Appendix PPP—Chief of Engineers, 1908, p. 2655.

Mr. Sellew. has come at which the only economical way to maintain the levees is to hold the banks against caving—not everywhere, but in all places where the special conditions exist which have just been described. This is a distinct stage of the work only recently reached."

The area protected by the Colorado levees will be occupied by farms of rather small area, and consequently a comparatively large population will be dependent on them. Intensive farming will be practiced, and the preparation of the land will be expensive, therefore it appears that overflow should be prevented by all possible safeguards. The integrity of the levee system will depend principally on bank protection. There are two general methods by which this may be accomplished: (1) to give the bank a revetment which will protect it against erosion; (2) to keep the main current away from the banks, so that erosion cannot occur.

Revetment may be continuous throughout the river, or salient points which are strongly attacked may be treated first, the work being extended as occasion demands. The most extensive revetment in the United States, if not in the world, is to be found along the Mississippi River and its tributaries, where the General and State Governments and local levee boards have engaged in protective works for many years. The standard revetment consists of a woven brush mat or fascine mattress protecting the bed of the stream for a short distance from the shore and the side of the channel below low water. Above low water, where brush deteriorates rapidly, the bank is graded to a uniform slope and paved with rock. Such work is very expensive.

The magnitude of bank protection and river control on a large stream is seen in a "Report by a Special Board of Engineers on a Survey of the Mississippi from St. Louis to its Mouth,"* with a view to obtaining a channel 14 ft. deep and of suitable width. On page 52 it is stated that up to the date of the report, 1909, there had been expended by the General Government on the Mississippi from the mouth of the Missouri to the head of the Passes more than $88 000 000. In addition, there had been some millions spent by the riparian States and local levee boards.

Reference is again made to *Occasional Papers No. 41*, Engineer School, U. S. A., being "A Résumé of Operations in the First and Second Districts of the Mississippi Improvement, 1882-1901." This material, which covers the work done by the National Government under the direction of the Army Engineers, has been compiled in detail by E. Eveleth Winslow, Major, Corps of Engineers:

(From page 45): "It is now considered settled that in a plan for the permanent improvement of the channel by the contraction of its low-water width, the work of bank protection should precede the permeable dikes and other structures of that class, and that the latter

* Doc. 50, 61st Congress, 1st Session.

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FIG. 79.—SIZE OF ROCK USED FOR PROTECTION.

FIG. 80.—LEVEE PROTECTED BY ROCK.

FIG. 81.—STEAMER "SEARCHLIGHT" DRAGGING LARGE ROCKS FROM CARS.

may be regarded as supplementary in functions and as necessary only in a minority of cases." Mr. Sellew.

(From page 47): "It was recognized from the beginning of the Commission's work that the construction of works of bank protection and channel contraction in detached reaches would be subject to influences more or less unfavorable from changes in unprotected bends above. In commencing work in the detached reaches of Plum Point and Lake Province, the Commission was not oblivious of this fact, but was influenced by other considerations which seemed to overbalance this disadvantage, and it has not since seen good reason to doubt the wisdom of that decision. But the experience gained in those reaches emphasized the fact that it is highly important in any project for general and permanent bank protection to leave no unprotected reaches above the work done from changes in which new conditions may be produced in improved parts of the river. And it is considered by the Commission that in undertaking a complete and permanent system of bank protection the work ought to begin at the upper end of the river at or near Cairo and be carried thence down-stream with substantial continuity. * * *

"It is considered that the annual cost of maintenance of completed work could not be estimated at less than ten per cent of the original expenditure."

(From page 233): "The object of bank revetment is to protect the banks of the river from destruction by the currents. The active agency in this bank destruction is erosion, and bank revetment must therefore prevent this erosion. Perhaps the easiest and most obvious method of doing this is by laying over the exposed surface of the erodable bank a non-erodable covering, and this is the method adopted. This non-erodable covering must, of course, be fairly durable, must have sufficient strength to withstand any strain put upon it, and must be free from interstices through which scour might take place.

"Owing to the fluctuations in the river, the different parts of the bank are subjected to different conditions. Below the low-water line the bank is always wet; above that line sometimes wet and sometimes dry, and this difference in conditions allows, if it does not require, that the portion of the bank above and below low water be treated differently.

"Above low water the bank can be seen and the non-erodable covering can be laid with ease, and it has been found that here a properly laid stone pavement forms an efficient protection. The individual stones of this paving must, of course, be so large that they cannot be moved by the current, and it was found by experience that stones of this size are not of themselves sufficient, as through their interstices scour could take place when the bank was sandy and the current strong. This has been prevented by first covering the bank with a layer of spalls or crushed rock of such thickness and closeness as to prevent scour through them and to hold this down with a layer of larger stone. Such protection, when carefully laid, has been proven to be all that is required for the upper bank.

"Below low water the conditions are different. As it is impossible to construct the non-erodable covering actually in place, the next

Mr. Sellew.

best thing is done, constructing it on the surface of the water above where it is to lie, and when constructed sinking it. In this way the continuity of the covering can be assured, and it can be placed exactly where it is needed. As has been stated, this covering must not contain interstices through which scour can take place, for in such a case not only would the purpose of the covering not be fulfilled, but by such scour its own eventual destruction would be assured.

"The covering actually used has been a mattress of brush and the only kind that has been found to prevent scour in very rapid currents is the fascine mattress. In moderate currents another type of mattress might do, but as changes in current conditions are always likely to happen the fascine mattress only should be used."

(From page 256): "In the early work brush was used as a covering for the upper bank, but, subject as it was here to alternate submergence in the water and exposure to the atmosphere, it decayed rapidly and possessed little strength after about two seasons, and it was this rapid decay of the brush permitting it to be easily broken up and exposing the bank to erosion, that led to its disuse and the substitution for it of an all-stone covering."

(From page 260): "It is probably safe to estimate that a fascine mat built of small brush will be effective for at least twenty-five years and that when a good proportion of large brush is used its life will be considerably extended."

Applying these principles at Yuma, revetment should begin at Laguna Dam and proceed continuously down stream, leaving no unprotected caving banks above completed work.

The portion of the revetment lying above low water should be constructed by grading the bank to about a 3 to 1 slope and paving it with rock. This is necessary, as any brush work laid on this portion of the bank, being subject to alternate periods of submergence and exposure to the air, will deteriorate in about two seasons and be of little use in holding the bank. Below the low-water mark, the sides of the channel and the bottom for 50 ft. from shore should be protected by a fascine mattress. Assuming that the gauge oscillation along the stream is about 12 ft., a slope of 3 to 1 for this height would give an exposed bank of about 38 ft., and, with a stone paving 2 ft. thick, would require 76 cu. ft., or, say, 3 cu. yd. of rock per lin. ft. of bank. By using the quarries at Laguna Dam and Pilot Knob, delivering the stone into barges, and then towing the material into place for use, a fair estimate for the material in place on the bank will be about $2 per cu. yd., or $6 per lin. ft. of bank. In addition, assume that brush mattress will cost about $4 per lin. ft., making a total for revetment of $10 per lin. ft. of bank, or $50 000 per mile. The distance from Laguna to the Mexican boundary is about 45 miles, and probably at least two-thirds of this distance, or 30 miles, will require continuous revetment. At a cost of $50 000 per mile of bank

or $100 000 per mile of river, 30 miles of revetment would cost about $3 000 000. Mr. Sellew.

The foregoing quotations from the Mississippi reports state that the annual maintenance of bank revetment should not be taken at less than 10% of the original cost, which means that expenditures for this purpose must be duplicated each 10 years.

S. Waters Fox, M. Am. Soc. C. E., an assistant engineer in charge of some of the Missouri River improvements, states* that 45 miles of the alluvial portion of that stream had been improved at a cost of $2 500 000, or about $56 000 per mile. The division of this cost was stated by him as follows:

Actual construction	67	per cent.
Care of, repairs, and moving plant	22	" "
Administration	9	" "
Other items	2	" "
	100	per cent.

It is thus seen that, in this particular case, the cost of maintenance was more than double that assumed for the Colorado. Another thing to be considered in the matter of bank protection is the source of the material to be used. The supply of rock in the quarries at Laguna Dam and at Pilot Knob is abundant, but the same cannot be said regarding the brush, which for the best results, should be young, live willow. In *Occasional Papers No. 41,* Engineer School, U. S. A., previously referred to, the following appears on page 46, in an extract from reports of the Mississippi River Commission:

"Another fact now definitely settled is that in the construction of mattresses of the type now employed on a large scale, the supply of material becomes an element of the problem presenting serious difficulty. The area of willow-producing bar necessary to supply material for a mile of fascine mattress 300 feet wide is practically five times as great as that required for an equal length of mattress built in the early years of the work. The quantity of small willow growth used in this form of mattress is so great that the estimate of the yearly supply available along the whole length of the river from Cairo to Vicksburg is not more than sufficient for the making of 15 miles per annum of bank revetment."

The writer is not aware how the growth of willow along the Colorado compares with that along the Mississippi between Cairo and Vicksburg, but will assume for the present that the quantities are about equal. The assumption will also be made that the quantity required for the Colorado will not be more than one-third as much as that on the Mississippi; therefore, if 600 miles of river provide for 15 miles of revetment on the Mississippi, it would provide for 45 miles

* *Transactions*, Am. Soc. C. E., Vol. LIV., p. 280.

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on the Colorado. If 600 miles of river will furnish material for 45 miles of revetment, then 45 miles of river between Laguna Dam and the International Boundary would provide for about 3½ miles of such work annually. In the 30 miles of stream to which it is suggested the revetment be applied there would be 60 miles of revetted bank, and at 3½ miles per year it would require 17 years to build this revetment with the ordinary growth. This makes no allowance for the annual repairs, which are estimated at 10 per cent. It would also be impracticable, even with the material available, to construct this revetment immediately, not only on account of the constructive difficulties of doing so much work in a short space of time, for the control of the stream by revetting its banks and contracting its channel must necessarily be slow because the changes which these works bring about cannot occur immediately. On this point the writer quotes from *Occasional Papers* previously mentioned, page 41:

(From the Mississippi River Commission's report for 1889): "This work is still incomplete, although its feasibility and value have, it is thought, been fairly demonstrated. It is, however, well understood now that it is likely to prove a slower process than was at first supposed, for the work cannot be hurried. It must be gradual and progressive."

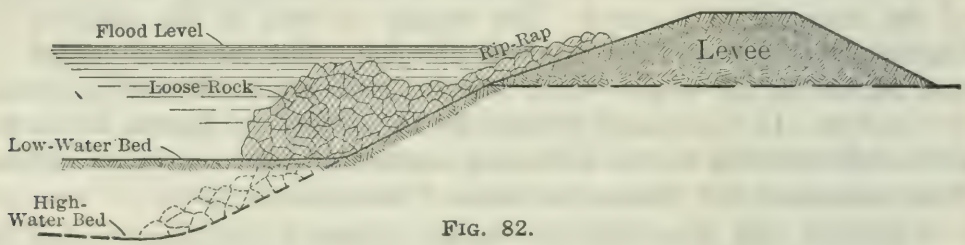

FIG. 82.

Seventeen years is entirely too long a period over which to stretch this work, as the protection of the Government's investment and the welfare of the settlers demand that the control of the river be almost immediate.

The radical difference between the high-water and the low-water channels of the Colorado furnish another obstacle to the foregoing form of bank protection. An examination of the diagram (Fig. 72) shows, that while protection of the upper bank by the rip-rap could be readily accomplished, it would be impossible to place the fascine mattress in the proper position. At low water, when conditions are favorable for mattress work, the channel is so silted up that a mattress sunk in position would not even approximate the place it should occupy to protect the flood channel from erosion. At high water, when the channel is scoured out and the bank to be protected is exposed, the depth of water and the velocity of the current are too great to accomplish work of this class successfully.

From the foregoing it appears that mattress revetment below low water, combined with rock revetment above, should not be applied along the Lower Colorado, for the following reasons: Mr. Sellew.

(1) The high first cost and excessive cost of maintenance (10%);
(2) The time required (17 years) for the natural growth to produce the willows necessary for this form of protection;
(3) The impossibility of placing the mattress in its proper position, due to the silted condition of the stream at those seasons when depth and velocity are favorable for such work; and
(4) That in great floods a concentrated attack might destroy a portion of the mattress at a time when its repair or the protection of the bank thus exposed would be impossible.

Rock Revetment.—Another method of protecting the bank would be to make the revetment entirely of rock. At low water the exposed slope would be graded, properly rip-rapped, and at the foot of this slope a large quantity of loose rock would be placed (Fig. 82). As the high-water channel was created by the scour of the current, this loose rock would be undermined and gradually slide into position, making a revetment along the bank thus exposed which would be practically a permanent protection. In approximating the cost of such work, let it be assumed that the average depth along the levee during high water will be about 20 ft. It is known that at exposed points where severe cutting has been in progress it has been as much as 32 ft., but, for a general average depth to which revetment should extend, 20 ft. appears to be safe. Suppose that the rock will stand at a slope of 2 to 1; then the length of the revetted slope in contact with the water will be 45 ft., and, if the revetment is 2 ft. thick, the area of the section will be 90 ft. Assuming that, owing to wasteful methods by which this is applied, one-half is lost, then the sectional area of rock per linear foot which has to be placed along the levee is 135 sq. ft., or 5 cu. yd., or 25 000 cu. yd. per mile of bank, equal to 50 000 cu. yd. per mile of river. For 30 miles this would mean the placing of about 1 500 000 cu. yd. of material.

Should such a course be adopted, quarries would be opened on each side of the river at Laguna Dam, which, by connecting railroads, would supply the levees within economic reach. Another quarry at Indian Hill, opposite Yuma, would serve for the portions above and below for proper distances, while quarries at Pilot Knob would control the remainder on the California side. From this latter point, a temporary trestle, which appears to be the best arrangement, would connect with the Yuma Valley levee, on the Arizona side of the Colorado. Later, if it was decided to extend similar treatment to the Gila levees, the Arizona quarry at Laguna Dam would control the levee on the north side of the stream, and the south levee could be reached from

Mr. Sellew.

Penitentiary Hill. Considering now only the embankments along the Colorado, it appears that the suggested treatment will require a railroad over their entire length, which is about 60 miles. These levees are already in existence, and the cost of equipping them with track (52-lb. rails) is estimated as follows:

Cost per mile of railroad on existing levee:

Steel, 52-lb. = 90 tons, at $30	$2 700
Fastenings	600
Ties, 2 600, at $1.25	3 250
Track laying	400
Train service	125
(Say, $7 000 per mile)	$7 075

There is a railroad on the Reservation levee between Yuma and Laguna Dam, and deducting this length of 13 miles from the 60 miles required leaves 47 miles to be built. At $7 000 per mile this cost would be $329 000, to which may be added about $10 000 for the Pilot Knob trestle, giving a total for railroad construction of, say, $340 000. The average length of haul from these quarries will approximate 15 miles, and it should be done for not more than 1½ cents per ton-mile. The 1 500 000 cu. yd. will equal about 3 000 000 tons, and with an average haul of 15 miles, gives 45 000 000 ton-miles, which at 1½ cents equals $675 000. Using large steam shovels, quarries with long and high faces, mined with small tunnels in which heavy charges are used, it would appear that the material could be placed on the cars at an outside price of 30 cents per cu. yd.; to this should be added, say, 25 cents per cu. yd. for spreading and placing the rock on the slope, giving 55 cents per cu. yd. as the cost above that of hauling. At 55 cents, 1 500 000 cu. yd. would cost $825 000, which, added to the expense of hauling, makes a total charge of $1 500 000, which is increased to $1 840 000 when the cost of building the railroad is included. Adding 10% for contingencies and repairs gives a round sum of $2 000 000 for protecting 30 miles of river, or about $66 000 per mile, equivalent to $33 000 per mile of bank.

Although these estimates must be considered as approximations, it appears that permanent rock revetment can be had for not more than one in which mattresses are used, and probably for less. The permanence of the rock, its slight maintenance cost, the ample quantity available, and the fact that it can be placed as well during attack at high water as at low, makes it the best and most economic protection for the banks of the river between Laguna Dam and the Mexican boundary. Once the tracks were on the levee, exposed points could be quickly protected, and the remainder of the work could proceed as desired.

By Deflecting Currents.—Bank protection by deflecting the currents away from the shore so that erosion cannot occur may be done by spur-dikes or by opening channels across sand-bars which are the cause of the impingement. The deflection of currents by brush dikes has been tried in many localities and with varying degrees of success. They are subject to destruction by the natural course of deterioration, by deep scour which occurs around their outer ends, and by the undercutting of eddies below them which they have created. They have been tried sufficiently along the Lower Colorado under the writer's direction to demonstrate that at best they are only a temporary expedient. If built of cottonwood piling, which grows along the banks of the stream, their life would not exceed 2½ or 3 years as the maximum; probably the average is not more than 2 years. Constructed of fir piling, they could be depended on for 6 or 7 years. If, at the end of that time, the new banks which had been created by silting between them were not protected by revetment, it is probable that the dikes would need renewal, in order to hold the work. One other disadvantage of spur-dikes is that the proper location for them is demonstrated only by the varying attack of the river, and, when this attack comes on, the water is generally so deep and the currents so swift that the dikes cannot be built. It is then necessary to defend them as best one can until lower water, and perhaps, on the falling stage, the river changes its point of attack, so that dikes are needed in other places. In the writer's judgment, protection by spurs should be used only in an emergency; they have no place in any permanent form of bank protection on this river. Mr. Sellew.

Dredging.—There remains the method of diverting the current by opening up channels across sand-bars. Wherever the banks have been attacked, the impingement has been caused directly by the formation of deposits on the opposite side of the stream. As these deposits build out toward the opposite bank, the flowing prism of the river is narrowed, the concentration of the water increases the velocity, the channel is correspondingly deepened, and, as a result, there is deep water and high velocity against a caving bank. Under such conditions, if it were possible to open up a channel of sufficient dimensions across this bar, and if the current could be induced to follow this new channel, the attack on the bank would be immediately relieved. In some instances, where the encroaching bar is short and other conditions are favorable, channels have been opened up and relief secured by a liberal use of dynamite. Such a method is impracticable in most cases along the river. It is seldom that a condition occurs that can be relieved by explosives.

The following is taken from *Occasional Papers No. 41,* page 44:

"The experience with the revetment work previously done had shown it to be inadequate for the purpose, and rapid changes of type

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were not made until there was developed the standard of the present day: A fascine subaqueous mattress with upper bank paving of stone. But as the revetment work had gradually been increased in strength, it had also increased in cost, until in the new standard type the cost had reached about $30 per linear foot of bank, or approximately $150 000 per mile.

"As the large cost of a systematic improvement of the whole river became evident, the Commission was led to investigate means for temporarily improving navigation during the low-water season. The principal obstructions met with are the sand bars built up during high stages, through which the river slowly cuts a channel as the flood recedes. If this process could be assisted and hastened, navigation would be improved thereby, and to this end experiments were made with dredges. These experiments, started in 1894, were successful and as a result the Commission was led to reconsider the whole subject of the river improvement, and as a result of this study the Commission decided to adopt a new plan for future operations, namely, to depend for the relief of the immediate needs of navigation upon dredging, and to restrict the improvement by bank revetment to special localities, where local interests or necessities demanded it."

(From page 46): "From all these facts together it became apparent several years ago—not all at once, but gradually, as facts accumulated and the deduction of experience was unfolded—that it was not possible by any or all of the methods which had been employed to accomplish such permanent improvement of the river from Cairo down as was necessary to meet the urgent demands of commerce within any reasonable time to come. It was this disagreeable but unavoidable conclusion from the work of preceding years, together with the recent and wonderful development of the hydraulic dredge, that led the Commission to undertake the experiments of dredging the bars of the river with the view to the temporary improvement of its low-water channel which has been detailed in recent reports of the Commission and to which more extended reference is made elsewhere in this report. Those experiments have proceeded favorably so far as to hold out a reasonable probability of entire success."

A suction dredge of large capacity and light draft which could be maneuvered in the Colorado during flood stages would be of value in opening channels across sand-bars. During low water the dredge could be used in creating a channel about midway between the levees at those points where attack from caving banks was imminent. The excavated material could be deposited in the vicinity of the levee through pipes carried on pontoons, thereby building the banks and making their alignment more regular.

It might appear at first thought that the quantity of material to be moved would require a dredge of unheard-of proportions, and that the expense would be prohibitive, but such work appears to be well within the range of the hydraulic dredges as developed for work on the Mississippi River, the largest of which have a capacity of 2 000 cu. yd. per hour. About one-third of their time is lost in tie-ups for

repairs during high water, maneuvering for position on bars, and in change of position, so that throughout the year the dredge would operate not more than 240 days, or 5 760 hours. Such operation of the dredge would result in removing about 11 000 000 cu. yd. of material. Mr. Sellew.

Assuming a low-water discharge of 4 500 sec-ft., passing Yuma with a velocity of 3 ft. per sec., the area of the flowing prism would be 1 500 sq. ft. If this was excavated, with a depth of 5 ft. and a width of 300 ft., the 45 miles of such channel between Laguna Dam and the Sonora line would contain about 12 400 000 cu. yd. It is clear that no such quantity of excavation will ever be required along the lower river in any one season, therefore the dredge under discussion has ample capacity. The questions to be examined are: (1) its cost; (2) whether it can be designed of dimensions which will allow it to navigate the shallow and tortuous channels of the Colorado; and (3) whether it can be operated during floods for opening channels across the bars.

Referring to Document 50, 61st Congress, page 55, where is shown a memorandum of a project for maintaining by hydraulic dredging alone a navigable channel, 14 ft. deep and 500 ft. wide, between St. Louis and the mouth of the Ohio River: The estimates provide for 20 dredges, each with a capacity of 2 000 cu. yd. per hour, to cost, with all attendant floating plant, tools, auxiliaries, etc., $6 000 000, or $300 000 per dredge. The annual maintenance is stated as $100 000 per dredge. In estimating the cost of a dredge, its life is here assumed as 20 years, and a sinking fund is provided for its renewal at the end of that time.

Estimated Annual Cost of Dredges.

Interest at 3% on $300 000	$9 000
Annual maintenance	100 000
Sinking fund	11 166
	$120 166

The maintenance of the channel by dredging would require continuous work, year after year, no permanent results being obtained. In about 16 years the sum of the annual maintenance would equal the estimated cost of the rock revetment, and the maintenance, capitalized at 3%, gives an amount double that for which it appears a safe and permanent bank protection may be obtained. Whether or not a dredge can be built to satisfy the local conditions cannot be stated from the data at hand, but as such operations will cost double those necessary for permanent bank protection, further examination in this direction appears to be needless.

Mr. Sellew.

CONCLUSIONS.

The purpose of this discussion is to indicate in a general way the present conditions along the Lower Colorado, and to point out, from an analysis of the data here assembled, the future course which will probably be productive of the best results. It appears:

(1) That in their present state, the levees are at the mercy of the meanders of the stream, except throughout those portions where a railroad occupies the levee top;

(2) That checking the meanders of the stream by permeable dikes during low water and brush defences during high water are simply emergency works which accomplish no permanent good;

(3) That owing to the large amount of scour during freshets and the fill-back on the falling stage, bank protection by means of willow mattresses as constructed in other places is impractical, if not impossible;

(4) That bank revetment, permanent in character and reasonable in cost, when compared with other methods, can be readily and quickly accomplished with rock placed from railroad trains operated on the levee tops and connecting with existing quarries which are within a reasonable distance;

(5) That further investigation is needed before it can be predicted, with any degree of accuracy, what effect the possible storage of flood water on the upper reaches will have on the flood discharge and the meandering tendency of the lower river;

(6) That it is doubtful if the amount of control obtained in this manner will eliminate the necessity of bank protection, and, even if such control could be obtained, a number of years must elapse before it can be made effective;

(7) In the meantime, existing works will continue to be menaced, therefore protective measures of greater or lesser magnitude must continue. In view of this fact, and the probability that some caving banks will always exist, the money spent for their protection should be for permanent revetment.

Nothing in the foregoing should be construed as opposing in any degree the creation of storage works at the head-waters of the Colorado. All feasible sites should be carefully examined, their probable influence estimated, and a well-determined construction plan matured. Such preliminary action is necessary before the advantages to come from the complete development of this river can be realized.

The destiny of this stream should be controlled by engineers rather than by statesmen, as the author suggests.

Mr. Mead.

ELWOOD MEAD, M. AM. SOC. C. E. (by letter).—The author's able review of the irrigation development in the Imperial Valley and the measures taken to control the Colorado River, describes the most

unique achievement in American engineering since Eads deepened the mouth of the Mississippi. The Society is indebted to Mr. Cory for having given such a complete history. Mr. Mead.

If the gap in the Colorado had not been closed, Imperial and a score of other towns would have been buried deep under the waters of an inland sea; the grain fields and orchards of thousands of farmers, and representing the savings of lifetimes, would have been blotted out; one-twelfth of the irrigated area of California would have disappeared; the Laguna Dam would to-day have probably been only a memory; and the greatest change in the earth's geography which has taken place within recorded time would have been an established fact.

Only those who heard the turbulent torrent that roared through the break in the bank of the Colorado and saw the forest-covered lands along the new channel melt and disappear in its depths, could understand the appalling nature of the disaster which impended and the tremendous odds which had to be faced to restore the river to its original channel.

In this attempt an expenditure of more than $1 000 000 had to be risked, with the chances on the side of failure. The courage with which Mr. Randolph and Mr. Cory assumed their trying responsibility, and the generalship which they showed in handling their immense equipment, are not made clear in Mr. Cory's paper. Only those who knew the conditions faced can fully appreciate the superb merit of their achievement.

The chief value of this paper, however, is not found in its description of the engineering methods used, but in the social and political conditions, attending irrigation development in the West, which it reveals. This history shows that, so far as this enterprise was concerned, there was an utter lack of public direction and supervision, or either State or National aid; and, as these are fundamental features of irrigation development, it is that portion of the paper which the writer proposes to discuss.

If it is the duty of the Government to conserve the public resources and to give any regard to the general welfare of the people developing them, this should be shown by the enactment of intelligent land and water laws and adequate administrative control of streams, as this is the only means of ensuring the full development of these resources and the right conditions of life in the arid West. This part of the country embraces two-fifths of the Nation's area. It was almost worthless without irrigation, it is immensely valuable now, but the change has been accompanied by a waste of effort, a loss of money, and the infliction of hardships and injustice which could have been easily prevented if the State and National law-makers had done their duty.

Mr. Mead.

When Western settlement reached the arid region, Congress should have provided for the diversion of rivers under public control, and in accordance with a general plan. This would have secured the irrigation of the best land, prevented the duplication of canals, or the building of canals where there was no water to fill them. It should either have built the large and costly irrigation works as Government projects, or safeguarded the private investments in these works, by making land and water security for the cost. Instead of this, there has never been any satisfactory title to water, and the public land has almost invariably passed into a control hostile to or independent of the irrigation works. Instead of adequate security, there was never, prior to the passage of the Carey Act and the Wyoming State law accepting this Act, any security for the money spent in irrigation works to water public lands, and at least three-fourths of these enterprises were financial disasters.

If all important irrigation works had been built by the Government, money could have been obtained at one-fourth the interest charge the majority of private projects had to pay, and without the ruinous discounts on loans which the flotation of these enterprises entailed. Low interest rates would have made much lower water charges possible and thus removed from American irrigation its most serious handicap.

If there had been any attempt to frame a land law suited to requirements in the arid region, it would have at once been manifest that the homestead should have been reduced in acreage instead of increased, because it requires more labor to cultivate 80 irrigated acres than it does 160 acres watered by rain, and the returns are larger. Instead of reducing the homestead and restricting settlement to homesteads, however, the Desert and Land Act was passed, which gave 640 acres to the individual, conferring title without residence, without cultivation, and too often with the mere pretence of watering. The Desert Land Act was promoted by the rich range stock owner. It was the worst handicap irrigation ever had, and, instead of aiding the deserving settler, it took away his best opportunities. Instead of public control of streams, men fought for their possessions with shotguns and shovels along the banks, or with more costly weapons in the Courts, where litigation, instead of settling issues, as a rule, created new fields for controversy. The water-right litigation of the arid States has not only been a continuing source of ill-feeling, injustice, waste, and loss, but its existence is a reflection on the capacity of the law-makers who are responsible for it. An understanding of these matters is necessary rightly to interpret what occurred in the Imperial Valley.

The irrigation and settlement of the desert of Southern California should never have been left to private enterprise. This was a case calling for direct Governmental control and responsibility. The desert

vas below sea level. It was worthless to the State, dangerous to travel- Mr. Mead.
ers, and ugly to look upon. It was known that the Colorado River vas a stream subject to torrential floods, and that the banks were lluvial; and it required only the slightest consideration to realize the langer which would menace settlers if they were allowed to make omes in this valley without the diversion works being made safe eyond question. The benefits which would come from irrigation and ettlement made it a matter of direct concern to every resident of California, gave the project a National importance, and justified its onstruction as a public work.

If, however, the State did not build the works, it was its duty to upervise them and protect the private enterprise which did. Before nvestors were allowed to risk their capital or settlers risk their lives nd comfort, some public authority should have seen to it that the and was fit to cultivate, and that there was a legal title to water; nd there should have been an inspection of engineering plans and an nvestigation of financial arrangements.

Instead of this, an irresponsible company without capital employs n engineer, whom it fails to pay. A right to 10 000 cu. ft. per sec. f a navigable international stream is established by the engineer osting a notice in an uninhabited jungle of weeds and brush. Of ourse, this was a farce, but it was the manner of establishing water itles prescribed by the law of California. That the title was worthess was shown later, but the early investors did not know this, they rusted the State to uphold and defend its statutes, and were betrayed. Any one who reviews this record must be amazed at the exhibit of mperfect laws and the incompetence or indifference of the public uthorities. The surveys of public lands were shown to have been raudulent, which was bad enough, but the delay of six years in correctng them was infinitely worse. The examination of the soil should ave been made before settlement instead of afterward, and should ever have been made except by experts having practical knowledge f irrigation. Instead of a proper land law, much of the land was cquired by non-resident speculators.

When the water title was attacked by the Interior Department, t was the duty of the State to defend its authority to grant titles and to rotect this struggling project and the unfortunate settlers. For this rotection, however, they had to look to a foreign country, and one vonders what would have been the result if Mexico had been as indifferent as the United States. This attack on the water title by the United States seems to have been morally wrong even if legally defensible.

Finally, when it was known that the river was beyond control, nd the most appalling calamity that ever menaced the State of California was impending, it should not have been left to a bankrupt company and impoverished settlers to cope unaided with this disaster.

Mr. Mead. It was the duty of the State or Nation to take charge, and provide the money and men needed to restore the river to its former channel. Apparently, no one in authority was interested, the State Government only considered the matter long enough to write a letter to the President, and the President, having Congress on his hands, shifted the responsibility to the head of a railroad company; and it was not until this railroad company took charge that we have the first refreshing example of generosity and public spirit. Nothing could have been finer than the action of Mr. Harriman. The loan of $250 000 when his time and resources were overtaxed by the earthquake at San Francisco, providing more than $1 000 000 for the last hazardous attempt to save the valley, furnishes an inspiring contrast to the supine indifference and irresponsibility shown by both the State and Federal authorities.

The Imperial Valley produced last year crops worth $10 000 000. The most desolate part of California has become attractive, productive, and profitable; but, when one contemplates the hardship, anxiety, and suffering of settlers, and the waste and loss to the investors in the enterprise, one cannot help feeling that the conditions which imposed such burdens on development ought to be removed.

The National and State benefits which have resulted ought to have brought both wealth and public recognition to the pioneers of the enterprise; instead, its engineer and organizer was in the end cast on the scrap heap, his life work wasted; and a large percentage of the pioneer settlers, after untold hardships, had to give up and leave. Many of those now in the valley have had to pay high prices for their land as the result of early acquirement by speculators. The Southern Pacific Company, although requested by the President of the United States to save this valley from destruction, has not been paid for doing so. The speculators in the company, who were astute enough to get two dollars for one, have in many instances become rich; the lawyers who have had charge of the unending litigation have reaped a fruitful harvest; but those who worked for the real success of the enterprise, the men who risked their own comfort and that of their families to create homes, have paid penalties which are entirely unnecessary where irrigation development is carried out under a proper system of public control.

Mr. Cory's paper needs to be read in connection with that recently submitted by Mr. Lewis,* the State Engineer of Oregon; both papers show the need for a change in the public conception of the duties and responsibilities of the Government in conserving and developing public resources, and a general appreciation of the need of more effective laws for safeguarding investments and making the settlement of arid lands easier, cheaper, and safer than this has been in the past.

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 637.

Mr. Follett.

W. W. FOLLETT, M. AM. SOC. C. E. (by letter).—Mr. Cory's paper is valuable and interesting for several reasons. It gives in chronological order the history of irrigation in the Imperial Valley, culminating in the terrible battles with the gigantic and treacherous Colorado; it records the impressions and beliefs, after sober second thought, of one who was there in responsible charge during the months and years of greatest stress, and, above all, it is especially pleasing to the writer because the author takes occasion to show how uncalled for was the hysteria which accused Mr. Rockwood of having made a criminal blunder in opening the cut through which the river made the crevasse.*

In March, 1903, the writer made a trip down the Colorado River by rowboat, starting at Yuma and going to a place known locally as "Mike's Landing," which was about 80 or 90 miles by river below Yuma and near the head of tide-water. The return trip was made by wagon, through the delta lands west of the river, along the west bank of Volcano Lake, and to Calexico.

On this trip, which was made on a Yuma gauge of 18.3, or about 1.5 ft. above low water for that season, it was noted that the height of the river banks varied from 5 to 8 ft. above the surface of the water. The maximum flood of 1902 was 24.5 on the Yuma gauge, and, judging from the high-water marks on the trees, filled the river a trifle more than bank-full, whether the banks were 5 or 8 ft. high. Apparently, so much water escaped as soon as the banks were topped that the rise ceased. After passing a point about where the crevasse of 1905 afterward occurred, where the width of the channel was 800 ft. or more, the latter narrowed, as water was spilled over the banks, until it was only 410 ft. between the overflow banks at Mike's Landing. These apparently did not overflow deeply—although at that time there had not been for many years such enormous discharges as have come during the last 8 or 9 years.

On leaving the landing, the wagon road led directly away from the river for about 10 miles and then veered to the northward, toward Volcano Lake. The high-water marks showed that there was a rapid fall in the ground as the river was left behind. Of course, these marks were not level, but fell away from the river, as drift showed that the current had been in that direction. The fall mentioned by the author —from 5 to 8 ft. in the first 3 000 ft.—was found, being nearer the latter figure. After that, the fall was more gradual.

In going north it was noted that each flood-water channel crossed was running on a ridge similar to, but much smaller than, that on which the main river ran.

As a result of that trip, the writer was impressed with the idea that the Colorado was nearing an epochal change in its course across its delta. It is well known that an alluvial stream builds up its banks

* *Transactions*, Am. Soc. C. E., Vol. LXII, p. 24.

Mr. Follett.

and beds across its delta until it lies on a ridge; and that finally the time comes when the river breaks away from this ridge, takes a new course to the sea across the lower portion of the delta, and proceeds to build another ridge. When the stream reaches this condition of instability it is impossible, except at great expense, to keep it on the high ground. The writer had no idea, however, that the river would break into the Imperial Valley, but believed that the ridge north of Volcano Lake and the embankment along the canal of the California Development Company, would prevent such a diversion.

Subsequent events have confirmed this idea, and the writer is now of the opinion that any money which may be spent in attempting to close the Abejas Crevasse will be wasted, because, even if it is closed and held, the river will break out again somewhere else.

The author has shown quite clearly* that no serious grade recession has resulted from the Abejas Crevasse. As the river has been running through this opening for 4 years, during which time there have come the two largest floods ever known, it is fair to assume that no further grade recession will occur, but that a new delta is forming, pointing toward Volcano Lake, and that the tendency henceforward will be for the bed of the river to rise. Therefore, no danger to the water supply of the California Development Company nor to Laguna Dam is to be expected. It is best, therefore, to recognize this as an epochal change, allow the river to take its own course to the Gulf, and protect the Imperial Valley by raising and strengthening periodically the levee which lies north of Volcano Lake and extends back to the river. For the valley this will be a sure protection, which the closing of the Abejas Crevasse would not afford for any length of time.

Attention is called to the narrow gorge between levees, which begins opposite Andrade and extends for 2 miles down stream. The levees average about 1 500 ft. apart, with a width of river channel of 1 000 ft.; that is, the average berm on each side between the river and the levee is only 250 ft., and, in some places, it is probably narrower. This condition is shown on Plate L.

If, during some flood, the current of the river should be deflected so as to strike the right bank, and scouring began, it is the writer's belief that the levee and canal bank would be eaten away like sugar, and the river would be diverted into the canal and down the Alamo, again reaching the Salton Sea.

To prevent this diversion, a safety gate can be built about 2 miles below the Hind Dam (see Plate L). At this point, the Rockwood Crevasse channel touches the sand hills on the north, and a cross-levee could be built from the gate to the main levee on the south. As there is much fall, a drop would probably be needed. In this might be

* Pages 1214–1215, and Fig. 3.

placed gates, if thought best, which could be used for sluicing sand from the canal above. Mr. Follett.

The writer was on the ground several times prior to the closure of the crevasse by the Hind Dam, and was strongly impressed with the danger of the river "side-wiping" the canal in this narrow stretch. If it starts to shift, the efforts of man to stop it will seem puny in comparison with the immense destructive force of the current. It is stated that, in the Edinger Dam, piles, 64 ft. long, were driven more than 50 ft. into the bed of the river, and that, when the flood struck them, they scoured out and floated away; they were not broken, but came out bodily. Attention is called to the author's statement (page 1214) that, in 1907 and again in 1909, it was found that for an increase of 10 ft. in the gauge height at Yuma there was a lowering of the bed of about 30 ft. This was where the river was flowing in a straight and unimpeded channel. It is not possible to say how deep it would scour in a sharp bend. It would be deep enough, however, to undermine and destroy the levee, regardless of any rip-rapping which might be on its face, or any rock which might be hastily dumped into the river. It is the writer's belief that this is a serious menace to the Imperial Valley. The fact that the river channel is now practically straight for about 4 miles is almost proof positive—bearing in mind the idiosyncrasies of an alluvial stream—that it will not remain in that condition very long. Whichever way the bend may start, the current will soon be attacking both sides, as it will rebound from one bank to the other.

R. H. Forbes, Esq.* (by letter).—To those who know the Colorado River, and who in a measure realize its eccentric and unmanageable character, Mr. Cory's account of control work in the Delta is rendered doubly interesting by that which the reader may find between the lines. For, in proportion as the work was difficult, original in methods adopted, and spectacular in its accomplishment, Mr. Cory's account is unassuming, matter of fact, and quietly scientific. Of the errors made by various agencies that in some way touched the reclamation of the Delta, he is most uncritical. Mr. Forbes.

An observer of events along the Colorado from 1905 to 1912 could not but be impressed by the efficiency of the Southern Pacific Railroad organization, with its ready and varied resources, its personnel, and its *esprit de corps,* in dealing with the situation. Not the least factors in a successful outcome were the democratic and friendly relations that existed between Mr. Epes Randolph, Mr. Cory, and others of the management, and the motley crew of Americans, Mexicans, and Indians whose confidence and loyal support they commanded to a remarkable degree.

Of the colossal and urgent nature of the task, the adverse weather

* Director, Agricultural Experiment Station, Univ. of Arizona, Tucson, Ariz.

Mr. Forbes.

conditions, the experimental character of the work, the conflicting advice received from influential sources, the diplomatic complications with Mexico, and the financial entanglements incident to the undertaking, Mr. Cory's own account is sufficient evidence.

In Dr. William P. Blake's account of the region, written shortly before his death, he relates the discovery in 1853 of San Gorgoño Pass, through which the Southern Pacific Railroad was built about twenty-five years later, and describes the descent of his party into the old lake bed, below the level of the sea.

He also outlines the topography and geology of the region, describing the lacustrine deposits, the fossils, the salt beds, the mud volcanoes, and to some extent the vegetation of the basin. He points out the manner in which the sediments of the Colorado, the tides, and rising land levels finally cut off a portion of the ancient Gulf of California, and describes the old lake Cahuilla thus formed, with its beach lines and its fluctuations in level as the river flowed to one or the other side of its delta. He even relates the Indian traditions of the former existence of this lake.

Most interesting, however, are Dr. Blake's quotations from his reports to the War Department in 1855, in which he points out the fertility of the soil, and the possibility of irrigating the region by a canal from the Colorado. He even states the danger of overflow through such a canal and the refilling of the ancient lake. The following words, quoted from the old report, have prophetic value:

"From the preceding facts [relating to soils and vegetation] it becomes evident that the alluvial soil of the Desert is capable of sustaining a vigorous vegetation." * * * "If a supply of water could be obtained for irrigation, it is probable that the greater part of the Desert could be made to yield crops of almost any kind. During the seasons of high water, or the overflows of the Colorado, there would be little difficulty in irrigating large areas in the vicinity of New River and the lagoons. By deepening the channel of New River, or cutting a canal so low that the water of the Colorado would enter at all seasons of the year, a constant supply could be furnished to the interior portions of the Desert. It is, indeed, a serious question whether a canal would not cause the overflow of a vast surface and refill, to a certain extent, the dry valley of the Ancient Lake."

Nothing is wanting here but a suggestion for the control of the river, as detailed in Mr. Cory's paper.

It is remarkable that a man in his early twenties, inexperienced in irrigation, and traveling for the first time through desert country, should have estimated so accurately the possibilities and liabilities of Salton Basin, fifty years before the development of the region. It seems that, until recently, Professor Blake's was not only the first, but the clearest, general conception of conditions in the Delta. Others

have promoted the irrigation of the Basin, and some have realized the possibility of a break-away; but, until 1905, additions to knowledge of the region were mostly supplementary to Professor Blake's descriptions. Mr. Forbes.

If one may look for useful suggestions in this connection, attention may be called to the necessity for a co-ordinated study, by the proper agencies, of our great physical problems, such as the utilization of a great river, and the proper control at all times of operations affecting the final solution of such a problem. The "pork-barrel" method of handling river improvements from the governmental side, is admittedly desultory and wasteful; and it is to be feared that scientific agencies engaged in the study of these same problems, as a rule, are not well co-ordinated. It may be that the Colorado has drawn attention to these facts, with corresponding advantages to come. The Newlands River Regulation Bill, now before Congress, will make possible a comprehensive study and development of the Colorado River as a whole, for storage, power, and irrigation; and the Carnegie Institution, on the purely scientific side, is soon to publish a collection of scientific papers relating to the Delta region, among which will appear Professor Blake's complete paper.

R. S. Buck, M. Am. Soc. C. E. (by letter).—The writer has read Mr. Cory's paper, as well as several of the discussions thereon, with great interest, but is forced to a view radically different from that of Mr. Sellew regarding the propriety or advisability of incorporating in such an engineering paper so much collateral matter of legal, financial, and (he might have added) political character. Mr. Buck.

Mr. Sellew says:

"The record of the operations incident to the return of the river to its former channel is extremely complete, although partly obscured by a mass of legal and financial entanglements which properly have no place in a purely engineering article."

Though it may have necessarily taken a longer time in the relating than the casual reader, in quest of purely technical knowledge, may have desired, one of the principal features of interest and value to the writer in Mr. Cory's paper is the graphic portrayal of the desperate odds which engineering talent and effort are often forced to meet, as a result of long-range financial management, clogging legal complications, and the blind, selfish play of the political game.

Engineering work is now controlled by forces far beyond the laws of physics, and, if he would occupy a position to which his technical talents properly entitle him, the engineer must learn something of these forces.

Mr. Buck. In the writer's opinion, Mr. Cory is to be commended rather than criticized for devoting so much attention to the conditions in question, for he has certainly thrown light on some dark places, and has shown how vital it is for engineers to consider, in their planning, the laws and habits of men and money, as well as the laws of Nature.

The Lower Colorado is not merely a problem of yesterday, presenting a vague general moral. It is a vital problem of to-day and to-morrow and of years to come. It will probably take a place alongside the Lower Mississippi River as another engineering problem, dragging out a long, weary, and costly existence as a result of Government incompetence and political selfishness.

Of course, Mr. Cory's paper can affect but little the thus far meager results of mighty efforts to control the Colorado, but it should have a marked effect on future efforts, if a reasonable amount of intelligence is to direct these efforts.

If Mr. Sellew's apparent views were shared by the Publication Committee, we would have no papers such as those on water laws and valuation of public utilities. The importance of these and similar collateral subjects, the writer feels, is growing to be more and more appreciated from the engineering point of view.

Mr. Robson. F. T. Robson, Assoc. M. Am. Soc. C. E. (by letter).—Under the heading, "Salton Sea", the author makes two statements which are quite noteworthy, and not to have been expected. The first is that the land uncovered as the sea recedes is being cultivated with entire success, and presumably with no preliminary treatment to remove soil alkalinity. This is the more significant because, before the formation of the sea, this land had an alkalinity so great that it was condemned by the soil surveys of the United States Department of Agriculture, mentioned elsewhere by the author. The soil alkalinity of this part of the desert is in general greater than throughout the portion first cultivated and for which the Imperial Area Soil Survey was made. Since then it has been covered for from 1 to 6 years by water which, according to Tables 20 and 21, was in 1907 more than one-tenth as high in soluble solids as ocean water (and generally similar in the proportion of the various elements), and is undoubtedly growing constantly stronger because of the submerged salt beds. Nevertheless, independent inquiry confirms the statement as to the successful cultivation of the land which is being exposed.

The other interesting and surprising statement is that the gross evaporation from the sea is only 6 ft. per annum. The writer has also taken the trouble to check this figure—and has found that it should be only 67 in.—and in doing so he has collected some information bearing on this point which it is thought will be of interest.

Mr. Robson.

The opportunity offered at the Salton Sea for careful determination of gross evaporation from large areas of water surface, and the relation of this evaporation to that from small pans and evaporators, was recognized by the officials of the United States Weather Bureau, and from October, 1907, to the end of 1910, a large amount of work was done there.

Special studies of evaporation loss from the surfaces of lakes and reservoirs were begun by the Weather Bureau, under the direction of Professors Bigelow and Marvin, at Reno, Nev., about August 1st,

FIG. 83.

1907, the main headquarters being transferred to the Salton Basin near Salton in the following October. Observations were also made simultaneously at the near-by points, Indio, Mecca, Mammoth, and Brawley, and at the following distant points: Phœnix, Ariz.; Eagle and Carlsbad, N. Mex., these being at low elevations in these States; Mitchell, Nebr.; Howell, Wyo.; Rupert and Boise, Idaho; Hermiston and Klamath Falls, Ore.; North Yakima, Wash.; Fallon, Nev.; Lake Tahoe, Cal.; on the Rocky Mountain Plateau up to an elevation of

Mr. Robson.

6 000 ft.; Birmingham, Ala.; Cincinnati, Ohio; Tupper Lake, N. Y.; and Lockport, Ill., in the districts east of the Mississippi River. Most of the western points selected were stations of the U. S. Reclamation Service, and at most of them evaporation observations were taken at 2, 6, and 10 A. M. and at 2, 6, and 10 P. M., though in a few cases observations were taken only at 6 A. M. and 2 P. M., which were found to be about the time of minimum and maximum evaporation, respectively, and from which, it has been shown, very accurate results can be obtained by taking the mean values.

The final bulletin on the data collected and the conclusions derived therefrom, though promised to be issued in 1911, has not yet appeared. It is hoped that it will not be much longer delayed. However, Professor Bigelow has given a summary of results and deductions in a preliminary report.* This preliminary statement does not contain the observed data, which, however, are abstracted by Professor Bigelow in the "Abstract of Data, No. 4", issued by the Weather Bureau, being "A Provisional Statement Regarding the Total Evaporation by Months at 23 Stations in the United States, 1909-10".

The preliminary experiments at Reno indicated that a large body of water loses only about seven-tenths as much as a small pan set in a dry place outside its vapor sheet. The further experiments indicated the loss to be near 65%, in the opinion of Professor Bigelow; and the results at the Salton Sea, properly corrected, in the writer's opinion, indicate the proper coefficient to be still lower, namely 60 per cent. This relation, from the practical point of view, is the most important fact brought out in the work at these various stations, particularly in that at Salton. At this point four towers were built: No. 1 about 1 500 ft. from the water's edge, No. 2 near the Southern Pacific trestle over Salton Creek about 500 ft. from the shore, No. 3 ¾ mile west, in 40 ft. of water, and No. 4, which is 120 ft. high, 1½ miles west, in 55 ft. of water (depths in 1909). These towers, though made as stiff as possible, were found to have a slight vibratory motion, even in quiet weather, and hence the water in the evaporation pans required to be damped in oscillation by submerged partitions.

The important results, as given by Professor Bigelow in the preliminary report referred to, are:

1.—That the wind velocity was so different at various heights, 0, 10, 20, 30, and 40 ft. above the ground—amounting often to 30% less at the ground than 10 ft. above—that every evaporation pan must be supplied with its own anemometer wherever evaporation observations are made, whether on land or on large bodies of water.

2.—That pans of different sizes evaporate at different rates, and

* *Monthly Weather Review*, February and July, 1910, pp. 307 and 1133, "Studies on the Phenomena of the Evaporation of Water over Lakes and Reservoirs."

these rates were found to vary approximately as the expression, 0.023 $(1.23)^n$, where n has the following values: Mr. Robson.

$n = 0$ for large open-water surfaces,

$n = 1$ for 6-ft. pans,

$n = 2$ for 4-ft. pans,

$n = 3$ for 2-ft. pans,

$n = 4$ for ordinary dry air.

This variation seems to be due to the banking up of the vapor on the leeward side under the action of the wind, and because the small pan clears much more completely of vapor, and evaporation takes place into a mixture of air of different contents according to the size of the pans and the force of the wind.

3.—The formula finally suggested by Professor Bigelow is (c. g. s. units):

$$E_d = C_2 \frac{e_s}{e_d} \frac{de}{d\,s} (1 + 0.070\ W),$$

where E_d = Evaporation in 24 hours (one day), in centimeters;

C_2 = Coefficient, found to be 0.138 $(1.23)^n$;

e_s = Saturation vapor pressure at temperature of the water surface, designated by s;

e_d = Saturation vapor pressure at temperature of the dew point in the air near the water;

e = General symbol representing vapor pressure;

E = General symbol representing depth of evaporation;

$\frac{e_s}{e_d}$ = Expression for the gradient (use Crelle's tables);

$\frac{de}{ds}$ = Expression representing the change of vapor pressure with temperature, s, taken. This quantity is merely the tabular difference in ordinary vapor pressure tables at the temperature under consideration (use Manual Table V).

W = Wind velocity, in kilometers per hour.

It is proposed to substitute this formula for that now in general use, which was proposed by Dalton in 1803 and may be written as follows (same notation):

$$\frac{dE}{dt} = C\,(e_s - e_d)\,(1 + aw),$$

where t = time, and C and a supposed by constant terms,

or $E_d = 0.036\ (1.23)^n\ (es - ed)\ (1 + 0.070\ W)$ using the values of the constants deduced from these experiments at Salton.

Mr. Robson.

4.—That water in pans of the same size evaporates at the same rates at 4 000 ft. elevation as at sea level, and hence the formula does not contain any barometer pressure term.

5.—The recommended formula has no annual period in the C_2 coefficient; the diurnal period is very small.

6.—A pan of very fresh water was allowed to evaporate alongside a pan filled with brackish Salton Sea water, which was gradually concentrated by adding water from the sea from November to May without emptying the old water. The brackish water evaporated a little slower than the fresh water, to the amount of 2% in April and May.

7.—The actual fall of the Salton Sea level from June 1st, 1909, to June 1st, 1910, was 51 in., and Professor Bigelow estimated that the rise was 12 in. due to inflow from the Alamo and New Rivers, and there were 6 in. of accretions from annual precipitation, making a total gross evaporation of 69.0 in. This total checked with results computed by formula from observed data necessary.

8.—Practically, it was the conclusion that engineers would necessarily have to adopt a standard pan and reduce the observed readings to the open-water surface, as explained above. Thus, the evaporation from a 4-ft. standard pan, when corrected for temperature and wind, and multiplied by 0.66, is very close to what observation suggests. If a water thermometer on a small raft in the lake measures s_0, and e_s and e_d are determined by using a sling psychrometer, through t and t_s, and the wind velocity by an anemometer as near the water surface as possible is W, then $C = 0.138$ for a 24-hour interval. In the formula, the mean values taken at readings made about 6 A. M. and 2 P. M. should be used.

The writer has checked the estimates of Professor Bigelow as to the inflow from the Alamo and New Rivers, and the increase due to the rainfall and run-off from the water-shed, from all the available data (of which considerably more exist now than when Professor Bigelow did his work). It is known that there are no springs or underground sources of supply discharging into the basin, because, until very recent years, the sea contained no water, and salt beds in the bottom were regularly mined. The rainfall is very light, consequently the run-off is small. From the time when the Colorado River was finally rediverted (the middle of February, 1907) to date is 6 years, and it seemed to the writer that the results should be checked to obtain an average for this entire period, rather than for the year from June 1st, 1909, to June 1st, 1910, which was taken by Professor Bigelow.

In determining the inflow, it is possible to utilize the sea itself as a large measuring tank in which to measure the combined rainfall on the water surface and the run-off from the drainage area due to rainfall—proper correction being made for the regular inflow from the

Alamo and New Rivers—by using the daily gauge heights or elevations of the water surface. Mr. Robson.

As a first step in obtaining the average evaporation for the entire 6-year period, the difference in elevation of the water surface on April 1st, 1907, and April 1st, 1913, was found to be 26.10 ft.

If, then, (1) the total precipitation on the sea itself, plus (2) the total run-off into the sea, plus (3) the total discharge of the Alamo and New Rivers into the sea for the same period, be added to this 26.10 ft., the result will be the total loss due to evaporation—percolation being considered as of no consequence.

a.—The writer prepared a table of the rainfall for each month of each year, as observed at Brawley, Calexico, Heber, Imperial, Mammoth, Mecca, and Salton, all these points being within the drainage area and fairly well distributed. It was then assumed that the total rainfall at all these stations, divided by the number of stations, would give a fair average of the precipitation on the sea itself. The total thus found for the 6-year period was 16.59 in. or 1.38 ft.

b.—A table was then made showing the decrease in elevation of the water surface for each month of the period, and the decrease for each month was compared with that for the corresponding month of the other years. It was found that the decrease in one month compared very closely with that for a corresponding month in another year, except in cases of unusual rains or other known causes. Eliminating these months, a total was then made of those remaining, and the result was divided by the number of months comprising the sum. The average thus found was assumed to be the normal average decrease for that month. Sufficient amounts, corrected for excessive rainfalls on the sea itself, were added to bring the eliminated months up to the average, and these additions were considered as the run-off or inflow due to precipitation. This total was 15.00 in., or 1.25 ft., for the 6-year period. The results obtained by this method were checked by examination of the daily precipitation records for these various Weather Bureau Stations. In such desert country, where the rainfall is scant, that which does occur is frequently in intense storms or in local cloudbursts. Monthly records, therefore, are of little value as indicating the probable run-off, and each storm at each observation station must be considered, instead of the monthly totals. Only two or three heavy precipitations extending over the entire water-shed occurred during the 6 years, the heavy rainfalls usually being rather local in character and in the nature of cloudbursts. The total number of such heavy precipitations throughout the period at all these stations, however, was surprisingly small, and the daily gauge heights of the sea surface for several days was carefully examined in connection with each of these heavy rains. In this way a very good check was obtained on the foregoing method of deter-

Mr. Robson.

mining the total inflow into the basin from its water-shed—exclusive of the New and Alamo Rivers.

c.—The discharge of the Alamo and New Rivers has been observed, and a rating curve has been applied only for a period from June, 1909, to October, 1910. Based on the records published in Water Supply Paper No. 300, and from information kindly given to the writer by Mr. Cory from his personal records, the following rises in the elevation of the lake surface have been determined, and are believed to be very close to the truth.

To January 1st, 1908								0.62 ft.
From January 1st, 1908,			to	January 1st, 1909				0.65 "
"	"	"	1909	"	"	"	1910	0.68 "
"	"	"	1910	"	"	"	1911	0.71 "
"	"	"	1911	"	"	"	1912	0.75 "
"	"	"	1912	"	"	"	1913	0.78 "
Total								4.19 ft.

To sum up, then, the evaporation has been as follows:

Loss in elevation of lake, as shown by gauge heights	26.10 ft.
(1) Total rainfall on lake surface	1.38 "
(2) Total run-off from rainfall	1.25 "
(3) Total discharge from Alamo and New Rivers	4.19 "
Total evaporation, April 1st, 1907, to April 1st, 1913	32.92 ft.
or a yearly average of	5.58 ft.
or	67.02 in.

as compared with 69.0 in., the total gross evaporation found and estimated by Professor Bigelow for the 1-year period, June 1st, 1909, to June 1st, 1910.

Table 31, containing observed data at Salton Sea, is taken from "Abstract of Data, No. 4," by Professor Bigelow, referred to previously. The figures for Nos. 1, 2, 3, and 4, for March, April, May, and June, were interpolated from curves plotted from records of previous years, and found by experience to give close results.

At the stations outside of the vapor sheet, at which observations were taken, for a 6-ft. pan on the ground, the total evaporation observed was:

Indio	119.33 in.
Mecca	107.81 "
Brawley	103.55 "
Mammoth	125.53 "
Total	456.22 in.
Average	114.055 "

TABLE 31.—TOTAL EVAPORATION, BY MONTHS.

NUMBER.	1.		2.		3.		4.		5.		6.		7.	
STATION.	SALTON SEA. TOWER NO. 1. 1 500 FT. INLAND.		SALTON SEA. TOWER NO. 2. 500 FT. AT SEA.		SALTON SEA. TOWER NO. 4. 7 500 FT. AT SEA.		INDIO, CAL. 10-FT. STAND.		MECCA, CAL. 10-FT. STAND.		BRAWLEY, CAL. 10-FT. STAND.		MAMMOTH, CAL. 10-FT. STAND.	
Position of pans, etc.	(1) Ground $D.$ = 2 ft.	(5) 40 ft. $D.$ = 2 ft.	(1) 2 ft. $D.$ = 4 ft.	(5) 45 ft. $D.$ = 4 ft.	(1) 2 ft. $D.$ = 4 ft.	(5) 45 ft. $D.$ = 4 ft.	(1) Ground $D.$ = 6 ft.	(2) 10 Ft. $D.$ = 2 ft.	(1) Ground $D.$ = 6 ft.	(2) 10 ft. $D.$ = 2 ft.	(1) Ground $D.$ = 6 ft.	(2) 10 ft. $D.$ = 2 ft.	(1) Ground $D.$ = 6 ft.	(2) 10 ft. $D.$ = 2 ft.
January	5.08	7.05	3.61	5.14	3.41	4.69	3.18	5.52	2.92	5.46	3.05	5.32	4.24	6.47
February	7.42	9.45	5.01	7.17	5.09	7.40	5.08	8.83	5.00	8.75	5.00	8.00	5.67	8.89
March	12.50	13.25	6.75	9.00	6.95	9.50	7.50	12.09	8.07	11.87	8.00	11.00	8.99	11.65
April	15.75	17.25	9.00	10.50	8.75	11.50	12.05	19.17	10.87	16.96	10.74	16.04	12.02	17.13
May	19.00	20.50	11.00	13.00	10.50	14.00	15.84	25.13	12.72	21.26	13.79	21.57	15.52	22.00
June	21.50	23.50	13.50	16.75	13.00	16.75	16.11	26.69	14.23	21.56	13.68	20.21	16.75	24.17
July	22.15	24.96	14.77	18.63	14.03	17.98	16.34	27.24	15.21	22.59	14.14	20.96	18.00	25.68
August	18.50	21.21	12.53	15.03	12.19	15.33	13.78	23.05	13.22	20.41	11.26	21.18	13.73	18.15
September	15.50	19.47	12.40	15.18	12.08	15.40	12.37	21.13	10.29	16.86	10.15	16.30	12.16	17.04
October	13.19	17.20	9.20	12.21	9.24	13.02	8.91	16.85	8.17	12.43	6.99	11.58	9.49	14.72
November	7.49	10.05	6.21	8.13	5.96	7.48	5.17	9.44	4.13	7.15	4.07	7.01	5.26	8.08
December	6.42	9.55	4.67	6.97	5.25	6.97	3.00	5.25	2.98	4.65	2.68	4.57	3.70	5.12
Year	164.50	193.44	108.65	137.71	106.45	140.02	119.33	200.39	107.81	169.95	103.55	163.74	125.53	179.10

Mr. Robson. The total evaporation, as determined by the writer, was 67.02 in. Hence the coefficient to be used in converting the observations from a 6-ft. pan on the ground outside the vapor sheet, and in a desert country, to the total loss from a large water surface, seems to be $\frac{67.02}{114.055}$, or 0.59, or, say, 0.6.

Mr. Chaffey. ANDREW M. CHAFFEY, ESQ. (by letter).—Careful reading of Mr. Cory's paper leaves the agreeable impression that he has sought to be fair and unprejudiced in handling a very complicated subject. Two other publications dealing with the Imperial Valley project have appeared, one mentioned by the author*, and another† consisting of short monographs by settlers, promoters, etc., gotten out by E. F. Howe, editor of the *Imperial Valley Press*. Both are rambling sketches, rather than histories, unsystematic, full of inaccuracies, and partisan and misleading statements, and hence untrustworthy and unsatisfactory. Mr. Cory's paper, therefore, is to be regarded as the first serious history of Imperial Valley, and will unquestionably be heavily drawn on by future writers. It is most fortunate, therefore, that it is being fully discussed, its inaccuracies pointed out, its engineering and historical matter supplemented by those qualified to do so, and, more particularly even, that several of the facts brought out by the author may be gathered together so that the reader interested in details may not fail to catch their true significance.

The writer was associated with his father, Mr. George Chaffey, in the management of the project from April, 1900, to February, 1902, and, for more than twelve years, has been collecting original documents and data relating to Imperial Valley. To these, doubtless much the most complete in existence, the author, unfortunately, did not have access, due to the fact that, curiously enough, he and the writer did not become acquainted until after the appearance of this paper.

On page 1251 the author states that the first practical step toward the irrigation of the Colorado Desert was the incorporation of the California Development Company in 1896. The writer feels that this is exceedingly misleading. Incorporating a company with large capital stock, appropriating vast quantities of water, by posting notices and recording the same, all at most nominal cost, acquiring options for small cash payments, getting out reports setting forth the merits of a project, and hawking the scheme about in financial centers, cannot rightly be considered as "practical steps" toward carrying out any enterprise. It is just this sort of "shoe string" promotion which Mr. Mead doubtless had in mind when writing his discussion on this paper (page 1509) and pointing out the folly of letting irresponsible

* "Born of the Desert," by C. R. Rockwood, 1909–10.

† "The First Decade," Imperial, 1910–11.

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companies go ahead without any public supervision whatsoever. On the contrary, the first "practical step" was taken when Mr. George Chaffey signed the contract (April 3d, 1900) to build canals into Imperial Valley. Many readers of the paper will undoubtedly get the impression from it that the California Development Company was at such time a going concern, and that its promoters had already produced tangible results. As a matter of fact, though it had been in existence nearly four years, it had as assets the two options on land—one in Mexico and one in California—mentioned by the author, as to each of which the Company was in default, and an idea, a project of large and attractive proportions on which not one shovelful of earth had been turned, and toward the consummation of which not one single practical step had been taken. Its obligations, aside from its capital stock, were various debts, aggregating a considerable sum, and $350 000 worth of what was called land scrip—issued prior to 1900 at 10 cents on the dollar and dissipated in promotion expenses—which was retired at face value in cash or in exchange for water stock of the various Mutual Water Companies at market price at the most inopportune time, namely, just before the water distributing system was completed in the valley.

The contract recited that the California Development Company owned, through a subsidiary Mexican Company, 100 000 acres of land in Mexico through which the canal must run. But for this representation (which turned out to be untrue) the contract would assuredly not have been made. The idea in reality was free to Mr. Chaffey, or to any one else for that matter. Indeed, Mr. Chaffey, at the request of Dr. Wozencraft, had considered the proposition in the early Eighties, before the later promoters had even heard of it.

To say, therefore, that at this date (April, 1900) there was a "practical change of management," that the engineering policy changed, or that "original plans were not carried out," is to create an absolutely erroneous impression. To quote Mr. Chaffey himself:

"I saw a way to accomplish the object with the means at my command by disregarding the Rockwood Survey altogether, adopting another line by which the bank of the canal would be made to serve as a levee to prevent the flood waters of the Colorado from finding their way into the Alamo, and utilizing many miles of that natural channel for the main canal."

This contract of April 3d, 1900, provided that Mr. Chaffey was to construct such canals as were necessary for the purpose of taking water from the Colorado River just above the Mexican boundary line to where the central main canal intersects the same boundary line in Imperial Valley, "said canals to have a capacity sufficient to deliver four hundred thousand acre-feet of water per annum at said last-named point"; that no more money should be expended "than is neces-

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sary in such construction"; that the construction cost of such canals should not exceed $150 000; that such construction was to be "at actual cost to first party"; that Mr. Chaffey was to receive $12 000 per annum salary for five years, payable when the Company was able to pay it, and one-quarter of the Company's capital stock; and that he should have charge of the Company's finances as well as its engineering works during that period.

It is felt that the foregoing abstract of this contract will suffice to indicate clearly its fairness, to say the least, to the Company.

From this date until the contract was terminated—April 3d, 1900, to February, 1902—Mr. George Chaffey and associates furnished the necessary money, and, dividing the engineering responsibility with no other person, designed and built the diversion works and located and constructed the main canal—including the Central Main west of Sharps Heading and in the United States—as it exists to-day. During this period he was assisted in the business management, in planning the system of Mutual Water Companies—which system, by the way, the author to the contrary notwithstanding, the California Supreme Court, in the case of Thayer *vs.* California Development Company, has recently not only held to be legal, but has highly commended—and in making the tri-party contracts, by the writer and by Messrs. L. M. Holt (who with Mr. Chaffey evolved in 1881 the first Mutual Water Company ever incorporated, at Etiwanda, Cal.), N. W. Stowell (who was the first to introduce concrete water conduits to irrigation construction in California in 1878, and who had large experience with irrigation works), and J. W. Swanwick, who was largely responsible for the tri-party contracts.

A great deal has been stated in the public press and in governmental reports—and Mr. Mead in his discussion (p. 1509) might easily be understood by the careless reader as coinciding therewith—condemning the California Development Company as an irresponsible corporation at all times prior to the Southern Pacific Company's taking over the management in June, 1905. This was far from being true during the period when Mr. Chaffey was directing its affairs—April, 1900, to February, 1902. The men just mentioned were those he associated with him; all except the writer were unusually experienced in the different phases of irrigation work and development, and financially able to build the canal. Mr. Chaffey himself was a man of wide experience gained in the practice of his profession of Civil and Mechanical Engineer. Best known as the founder of Ontario, Cal. (1883), he had founded Etiwanda, Cal. (1881), and had developed the irrigation systems in both places to such a state that the visit of a Royal Commission from Australia in 1885, resulted in his proceeding with his brother to that country and establishing the irrigation dis-

tricts of Mildura and Renmark, both at this time populous and prosperous communities, conceded by all public men to be "the great object lesson in irrigation for the rest of Australia." In 1882 Mr. Chaffey produced the first electric current (generated by water-power) in Southern California, and in 1884 lighted the City of Los Angeles with electricity. A large number of the first settlers in the Imperial Valley were actually from Etiwanda and Ontario; all were experienced irrigationists, and it is safe to say that 90% of them made their purchases because they personally knew of Mr. Chaffey's previous successes. Mr. Chaffey.

The writer, of course, agrees with Mr. Mead that the State or Nation should protect settlers on and investors in irrigation projects, examine and approve engineering plans, etc., etc., and especially he agrees with his comments regarding water right laws, the unfortunate condition of which is responsible for most of the financial disasters which have overtaken irrigation enterprises in the West, and of which the California Development Company is, perhaps, the most notable example. At the same time, it must be admitted that the greatest and most rapid irrigation development the world has known is that responsible for what is known as Southern California, and the Government—State and National—in no wise helped and not seldom added severe handicaps. It is sometimes said that Southern California has been built on "shoe strings"—certainly, many of its most successful enterprises have been started with inadequate capital, on confidence in the proponent's resourcefulness. Under such conditions, people are "live wires" to a degree beyond the comprehension of officials and employees of powerful corporations and of the State and National Governments. Much of the resentment toward, and often unfair criticism of, the U. S. Reclamation Service, by owners and officials of, and settlers under, private irrigation undertakings had its beginnings in the attitude of those who never need to consider for an instant financial arrangements, discounting securities, etc., toward "flimsy" work and the twists and turns of men driven to their wits' end when doing things with insufficient capital. It is well enough to say "Don't try them if you cannot get enough money to insure success." Such a policy would undoubtedly prevent not a few failures; but, on the other hand, by it the Southern California of to-day would not have been for decades to come.

The Imperial Valley is possibly the most striking example of this. In the first place, if the soil had been first examined by Government experts instead of by practical farmers, the project would have been condemned *in toto* and the region still original desert. Next, work was begun with small capital in hand, and in 10 months' time, and at the cost of less than $100 000, Mr. Chaffey put water into the burning desert 60 miles away from the Colorado River, and along lines, engineering, legal, and financial, which experience shows would have meant large success to all (had the Government only kept its hands

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off), and by doing so realized the visions of the dreamers and the hopes of the promoters who for half a century had dreamed and promoted, and accomplished nothing. In less than 2 years, the Imperial Main Canal, essentially as it is to-day—except the $75 000 head-gate at Andrade—and more than 400 miles of distributing ditches, were completed and the water was ready to turn on to 100 000 acres of land, at a cost to the settlers averaging less than $10 an acre; and the capital at hand was less than the promoters had expended in the hitherto vain effort to get started.

In contrast is the Government Yuma Project, with unlimited capital: $3 000 000 initial appropriation and more than $3 000 000 later; time, 8 years; acreage, 100 000; cost, $70 per acre; bitter disappointment and heavy loss to settlers relying on Government estimates as to money, and, more particularly, time required to complete the work. As a part of the Yuma Project, Imperial Valley would now be in exactly the state it actually was in 1902—eleven years ago—as far as development is concerned. In the meantime, in 1905, it was sending out $5 000 000 worth of products, and at present more than $10 000 000 worth. The gain in time has more than paid all the real damage *per se* done by the runaway river.

As the author has pointed out, this misfortune would never have occurred had it not been for numerous unnecessary and inexcusable obstacles. For some of these there is possibly some excuse, or at least explanation, but the soil survey and report was the most disastrous and inexcusable of all. The space devoted to it by the author in his paper is out of all proportion small compared to its significance, and the facts about it should be elaborated somewhat.

As stated by the author, a few samples of soil from this region were analyzed in 1893 by the Director of the California Experiment Station, University of California, Professor Hilgard, and again in 1896-97, and also in 1900, when the Imperial Land Company began active work. In the summer of 1901, the Bureau of Soils, U. S. Department of Agriculture, requested the co-operation of the promotion companies in making a study of and report on the soils of the Imperial Valley. This was gladly given, and, on October 17th, Mr. J. Garnett Holmes, of the Bureau staff, arrived and began work. Later, his immediate superior, Thomas H. Means, M. Am. Soc. C. E., and others of the Bureau, joined him, and on December 20th (in 63 days) the field work was completed. It took only 3 weeks (January 10th, 1902) to issue a preliminary report, "Circular No. 9," covering the 169 sq. miles of territory which had been examined. Before this circular reached California, an Associated Press dispatch from Washington advised that:

"The condition of soils in the Colorado Desert in San Diego County, as disclosed by a survey just completed by the Agricultural

Department in the vicinity of the new irrigation plant at Imperial, is reported to be 'much more serious than was anticipated.' This is the expression used by Milton Whitney, Chief of the Bureau of Soils, in a preliminary bulletin which he has deemed best to issue by way of warning." Mr. Chaffey.

This refers to the report which the author says was "unfavorable, * * * and calculated to deter sensible people from settling in the region." This is a more than conservative way to put this matter, as is shown by the single extract—by no means the strongest of many—which the author quotes on page 1271.

In 1903 the U. S. Department of Agriculture, Bureau of Soils, issued the "Soil Survey of the Imperial Area, California (Extending the Survey of 1901), Advance Sheets of Field Operations of the Bureau of Soils, 1903." This document, under the head of "Present and Prospective Development," on page 31, makes the statement:

"The sole dependence of the people of the Imperial Area must always be agriculture. * * * The source of wealth, therefore, is limited to the soil, and to a greater extent than in almost any other part of the country. Thus the problem confronting at least a part of the farmers in most arid regions is here—where alkali is so generally distributed in the soils and the conditions so unfavorable to reclamation—about as serious as it could well be.

* * * * * * *

"The people of the Imperial country should recognize the fact that aside from the general problem of securing water for irrigation they have to solve perhaps the most serious agricultural problem of the arid West. Here is found a most refractory soil, much of it impregnated with alkali. The only way to benefit the land is to carry away the salts. The application of gypsum cannot be of the slightest benefit. Little or no benefit will be derived from running water across the land with the expectation of flushing the alkali off the surface. * * * The water must pass through the soil and find ready egress through natural or artificial drainage ways.

"The quantity of salt taken out of the soil by alkali-resistant crops, such as sorghum and sugar beets, is not appreciable, the benefit to the soil in such cases resulting mainly from the prevention of evaporation at the surface through cultivation and from the leaching of the salts into the subsoil by irrigation water. If the subsoil be practically free from salt, as is often the case in arid regions, this vertical distribution of the salts alone is often all that is necessary so to reclaim the soil that sensitive crops can be grown. But it has been shown that in the Colorado desert the subsoil is the greatest source of danger, so that every bit of salt removed should be permanently removed. * * * By far the greater extent of the soil of the valley is a sticky, plastic loam or clay, through which water passes very slowly. The natural drainage in the soil itself is very poor, and to be anything like adequate it must be supplemented by artificial drains to carry off the ground

Mr. Chaffey. water as the salts are washed down from the surface. * * * Tile drains are the best and in the end the most economical for draining the land. They are, however, expensive, and the question is, will it pay?

* * * * * * *

"That it will eventually pay to drain these lands there can be little doubt, but in the present state of the country concerted action will be difficult and individual effort futile."

In February, 1902, the University of California, College of Agriculture Experiment Station, issued Bulletin No. 140, entitled "Lands of the Colorado Delta in the Salton Basin." This bulletin gave essentially the same results of soil analyses, but was much more careful in its comments, and on pages 50 and 51 gave crop reports from actual settlers which were excellent. It also stated, after giving a list of native plants from the Salton Basin, on page 44, as follows:

"The list of plants here given is notable for the absence of most of the species considered elsewhere as prominent alkali indicators. We miss at once the salt—or alkali—grass (*Distichlis*); the 'Greasewood' of Nevada (*Sarcobatus*) and that of the San Joaquin Valley (*Allenrolfea*); the samphire (*Saliscornea*); and the tussock-grass (*Sporobolus arioids*) * * *. But as a whole the collection made does not speak of 'irreclaimable' alkali land so far as we know their habits. * * * Taken as a whole, the native vegetation does not altogether confirm the unfavorable impression derived from the leaching of the soil samples."

In addition to these soil reports, in January, 1902, several interviews, written up in a sensational manner, appeared in the daily papers in California, practically condemning the Valley and virtually warning investors and settlers from the field. These were the most disastrous of all in their effect on the enterprise.

Experience has proved that these reports were fundamentally wrong in their conclusions, as less than one-half of 1% of the lands brought under cultivation has failed to produce most satisfactory, indeed, phenomenal crops. Nevertheless, the Government's publications were such that the author's statement: "It seems certain that, had the territory not been already settled in very large measure when these reports were sent out, Imperial Valley would yet be unreclaimed" (page 1271), is undoubtedly absolutely true.

The author is in error on page 1264 in referring to the elevation of the bottom of the Chaffey head-gate. This structure was started in March, 1901, and work was rushed as rapidly as possible. When completed, its floor was 3 ft. below the river bottom, and, to make the gate more secure, three 2 by 12-in. sand-boards were put on the toe of the **A**-frame, to be removed when the sand and silt became settled

around the structure; there was no necessity for removing them while Mr. Chaffey was directing affairs, and, later everybody apparently forgot about them and they were never taken out, the idea thus obtaining that the floor of the gate was at the top of the sand-boards when in reality it was much lower, and as low as it was possible to place it that season. Working at high pressure against a rapidly rising river, the head-gate was finished, and Mr. Chaffey himself turned the water through it on May 14th, 1901. Mr. Chaffey.

Before leaving this subject, and in view of the controversy that has raged regarding the necessity of the fatal cut in Mexico (made more than 2 years after Mr. Chaffey had severed his connection with the Company), it is interesting to note, from Government records, that the old Imperial Canal, from the old intake, carried throughout the Summer of 1904 and until October, 1904, when the Mexican Intake was cut, more water than ever before, and quite enough to supply the users in Imperial Valley.*

The author's statement (p. 1255) that the initial price charged for water stock was $5.75 is misleading. Those who purchased during the construction period were given bonds of the California Development Company equal in amounts to their initial payments, bearing 5% interest and acceptable at their face value for the last installments on their notes. Hence, those paying $3.00 cash on their water stock purchases got water rights at $5.75 per acre. Only $80 000, par value, of the bonds were thus given out to water stock purchasers on total sales of 50 000 shares in Imperial Water Companies Nos. 1 and 4, the settlers coming in having so little cash and speculators so little faith that they could not or would not take full advantage of the opportunity offered.

The author is again in error as to the reasons for the termination of the five-year construction contract between Mr. Chaffey and the California Development Company. It was not because the former was frightened out by the soil reports, but because it was found that the power of attorney to vote, for five years, stock, which, with his own, constituted a majority and control, was found ineffective on account of the certificates not having been actually turned over. Success had been achieved and these shares of stock began coming in for transfer. It became a matter of buy or sell, and the original promoter's idea of value was deemed to be too inflated, so Mr. Chaffey made the best terms possible for his holdings and withdrew in February, 1902.

The author's outline of the Delta Investment Company and its activities is also misleading, and in some respects entirely erroneous.

* See "Progress Report of Stream Measurements for the Calendar Year 1904," by the late W. B. Clapp, M. Am. Soc. C. E., Water Supply and Irrigation Paper No. 134, p. 27, and compare with measurements in previous years.

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The facts are that in the fall of 1901 it became apparent that the sales of water stock would not produce sufficient money for the work that must be done, and Mr. Heber organized the Delta Investment Company in the hope that by giving it a favorable contract to purchase the bonds and mortgage notes of the California Development Company, outside capital could be attracted to the new company. The Delta Investment Company was launched with the consent and approval of those owning or representing all the stock of the California Development Company. By the terms of this contract, the Delta Investment Company agreed to purchase bonds and notes from the California Development Company to the amount of $21 000 in cash per month for twelve months, thus guaranteeing the California Development Company $252 000 in cash, the amount estimated as absolutely necessary to spend during the coming year in a permanent intake head-gate, canals, and other necessary works. In consideration of this guaranty the Delta Investment Company had the right for 12 months to purchase any or all of the bonds and mortgage notes of the California Development Company at 50% of their face value.

As the whole enterprise was still considered by outsiders as an experiment, and the banks of Los Angeles refused to concede any value whatever to the Company's "securities" for loaning purposes, or to purchase them outright at any figure at all, and the soil experts had commenced giving out their unfavorable opinions, the only reasonable criticism of this contract is that it was unfair to the Delta Investment Company and, consequently, unfair to those stockholders of the California Development Company who were large stockholders in the Delta Investment Company, in that it indirectly threw on them the burden of financing the California Development Company without adequate remuneration.

That the privilege of buying the bonds and mortgages at 50 cents on the dollar did not prove especially attractive is best shown by the fact that only a small fraction of the holdings of the California Development Company changed hands under it during the 4 months it was in operation. Mr. Cory, therefore, is correct in stating (p. 1260): "It must be admitted that the Delta Investment Company took over such securities at a larger price than could have been obtained from any other source," but quite incorrect in stating either that there were "many" or "large" or "apparently dishonest" transactions between the two corporations, or in giving the impression that it was a hardship on the California Development Company to part with $2 in its so-called "securities" for $1 in hard cash. As a matter of fact, the California Development Company could well have afforded to be much more liberal, and the writer knows that if any one had come into the

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office at that time with $252 000 in gold coin and offered it in exchange for $1 000 000, face value, of the so-called "securities," there would not have been a voice raised against taking the money, it was so urgently needed.

Finally, the author is entirely mistaken in saying that March 1st of that year found the California Development Company "with all its bonds gone, its collateral notes and mortgages largely depleted, no money in the treasury, and deeply in debt." The facts are that at that date it had $47 000 of its bonds on hand, $525 000 of mortgage notes, $235 000 of accounts receivable, besides lands, canals, machinery, equipment, etc., and unsold water stock estimated (at the selling price at that time) to be worth $2 235 000, and its liabilities, including $453 000 of bonds and deferred payments to Hanlon and Andrade, were less than $500 000; in truth, from a few months after Mr. Chaffey started work, the California Development Company never was without an ample supply of mortgage notes and bonds in the treasury, but outsiders had not sufficient faith in them to buy them for cash.

In view of what was accomplished in the 22 months ending February, 1902, and the fact that some of the best men in Los Angeles would have joined him and furnished large amounts of capital, provided he had secured full control, it is doubtless safe to say that had he taken control, instead of retiring from the company, there never would have been any runaway Colorado River and the development of the Valley would have been even much more rapid than it has been.

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C. E. Grunsky, M. Am. Soc. C. E. (by letter).—The problems of the Lower Colorado River have a wonderful charm. They are presented by Mr. Cory in a most comprehensive and attractive way, and he has done the Profession valuable service in putting on record and making available for convenient reference the results of an experience on river work which is unique in character and magnitude.

So long as there was a fighting chance, in 1906 and 1907, the engineers under Mr. Cory's guidance and upon the wise determination of Mr. Harriman were to keep up the fight. How the fight was won has been well told by Mr. Cory. The writer takes pleasure in again expressing his admiration of the able way in which the work was handled.

At the outset, every one, so far as the writer knows, believed that the work of making a closure in the fall of 1906 would be greatly facilitated by the use of brush mattresses on which to dump the rock. What the work done on the Colorado in 1906 and 1907 has clearly demonstrated is that, with adequate transportation facilities, a brush mattress on which to drop the rock is not essential. It is only necessary to place the rock more rapidly than the water can undermine and bury it.

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Without the spur track from the Southern Pacific main line down to the Lower Mexican Heading, the first attempt to close the break would have been a failure, and yet the writer recollects distinctly that plans were under discussion for the beginning of this work, under which reliance was to be placed on water transportation. The outcome has demonstrated clearly that, when such work is to be done, every possible contingency should be foreseen so that every emergency can be met properly. It was a wise precaution, too, to keep an equipment and a force on hand and available for repairs during the high water of 1907, even though, as it turned out, they were not needed.

At that time the levees were new, and the river brought down an unprecedented volume of water—a maximum of about 100 000 sec-ft.* Naturally, it would be expected, therefore, that the levees had at once been subjected to a severe test. This might have been the case if the conditions on the lower river had remained as in years past; but they had been undergoing a rapid change, with the result that the new levee was only wet here and there, and there was no opportunity to observe what might have happened if the water had been 3 or 4 ft. up on the levee.

The writer made a special study of the problem of the Lower Colorado River for the Secretary of the Interior in the early summer of 1907, and his observations at that time and in the two preceding years as Consulting Engineer in the U. S. Reclamation Service, together with many years of familiarity with certain features of the river, are the basis of the comments which are now prompted by Mr. Cory's paper.

As early as 1883, or thereabouts, Gen. G. W. Shanklin, at one time State Surveyor General of California, and Mr. Duncan Beaumont called the writer's attention to the New River country, as a part of the Imperial Valley was then called, and the desirability of making an instrumental survey to see what could be done toward accomplishing the irrigation of that valley was discussed. It was known at that time that the only practicable route for a canal would be across Mexican territory. The situation appeared to be too complicated, and nothing was done; but the writer, from that day to this, has never lost interest in the problem.

In 1896 the writer explored a portion of the Colorado River below the Grand Canyon and confirmed the selection of a dam site at the lower end of Iceberg Canyon, as the best available, in a long stretch of river, for a low dam to be used in creating a fall for power. He reported at that time that it would be feasible to construct a dam of loose rock on a sand foundation by blanketing the bed of the river for a considerable distance up and down stream with broken rock, using large blocks for the down-stream portions of the work, allowing

* Erroneously estimated at 115 000 sec-ft. by the U. S. Geological Survey in Water Supply Paper No. 249.

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the water to bury these as deep as it would, and using finer material in the up-stream face, which would ultimately be made impervious or nearly so by the river silt. The plan of that day involved the complete turning of the Colorado over a low spur of the mountains. Mr. Cory's experience at the Lower Mexican Heading confirms the feasibility of carrying out work of this character.

Several years later the writer was stationed for three months on the Colorado a few miles above the mouth of the Virgin River, and had additional opportunity for observation. At that time boilers were to be set for mining operations, and brick was wanted. No clay deposit could be found. The most promising mud-bar in the river was sampled. The mud could be balled up and would crack somewhat in drying. It was submitted to a chemist at San Francisco and reported to be composed almost entirely of very fine silicious sand entirely unfit for brick-making.

Above the Virgin River the water of the Colorado, at low and high stages, carried a large quantity of this silt in suspension. This would settle quickly when the water was dipped from the river in a glass, and within a few seconds the water was clear enough to drink. There was certainly comparatively little clay in the silt carried by the river water at this point. This observation leads the writer to assume that the main Colorado must be the principal source of the fine, light sediment, other than clay, which the river carries to the delta, and accounts, too, for the preponderance of material of this character in the river delta; it also makes it appear probable that the clays, bedded as the author describes, probably have originated in other parts of the river's water-shed.

During the low stage of the river, above the Virgin, the writer had occasion to erect a mast or pole on a gravel bar for the support of a cable. The gravel was unusually firm. It consisted of all sizes from cobble-stones down to sand, and in its dry condition appeared to be almost cemented. The mast was erected and large boulders were piled around it. When the river rose the gravel around the mast became wet, softened, lost its cohesion, and acted very much like a quicksand; the mast required additional support or it would have fallen.

A heavy crankshaft lying above the water on another portion of the gravel bar commenced to sink into the gravel when this was wet by the rising water, and was with difficulty supported until it could be transferred to safer ground.

These facts are alluded to as a warning against placing too much reliance on gravel for the facing of bank or levee slopes. The gravel, in the case of the Colorado River levee, serves a most excellent purpose, and its use is not criticized, but gravel has no particular virtue as a material for resisting the attack of a current, and will have but little resisting power when used in spur dikes. It is an excellent

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material, however, as in the case of the levees on the Mexican side of the Colorado, for blanketing the levee, serving as a protection for the earth slope against wind erosion, and checking to a large extent the activities of burrowing animals.

The Colorado below Yuma, as explained by Mr. Cory and as noted by the writer in a paper presented to this Society in 1907, is flowing down the slope of a delta cone. While undisturbed by human agency its floods annually watered broad areas, and the rank vegetation which sprang up on these areas was an effective barrier opposed by Nature to the concentration of the river's flow along new channels, even though some of the water going over-bank should long ago, if topography alone is considered, have cut out a new course for the river. The river has been compelled by these restraining forces to hold for more than 500 years to the course as geographers first knew it from Yuma to the Gulf.* However, though the river, unaided, could not break away from this general alignment, it swung back and forth, within restricted limits, on an ever-changing serpentine course. The river along the thread of the stream has a fall of about 1 ft. per mile, and the average fall along the air line from Pilot Knob to the Gulf is about 1½ ft. per mile.

When attempts are made to confine a river of this character between levees there is trouble. Under the attack of the river, the banks will continue to break away, first at one point and then at another, and no amount of care in the location and construction of levees can insure them against destruction from this cause. To check the caving bank is difficult and expensive, even when adequate facilities are at command, as has been well set forth by Mr. Sellew. In ordinary cases, it is out of the question to eliminate entirely this source of danger to levees located comparatively near the river bank.

It is natural, then, that the levees in the case of such a river should be set far back from the banks. In this location on the Colorado the levee cuts off the river bends and, therefore, follows a line of greater slope than the river. This is actually the case below Yuma, except for the first long stretch below Pilot Knob, where the river is straight for some 8 miles and the levees on each side are parallel with the stream and comparatively near the banks. Below this straight reach, the river channel, down to the Arizona boundary, is about 30 or 40% longer than the levees on their present location (Plate LVI).

If the river rises above banks and water stands against these levees, some barrier must be interposed to its flow if erosion at the base of the levee is to be prevented.

Even when constructed as was the original levee at the Lower Mexican Heading, with borrow-pits on the land side, the work of levee

* *Transactions*, Am. Soc. C. E., Vol. LIX, p. 6.

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building gives more or less open space for the water on the river side along the levee, and there will be concentration of flow there which will rapidly cut channels in the light silt soil of the delta. The spur dikes, as built by Mr. Cory, offer some obstruction to this flow, and are of value under certain conditions; they have been of use during such moderate bank submersions as have occurred since the levee was built, but under a real test, with water 4 or 5 ft. on the bank, they would serve to concentrate fall and would start local cutting which might prove quite as dangerous as the general erosion due to uniform flow along the levee. It may be stated in a general way that the efficiency of such spurs for the same aggregate quantity of material in them will ordinarily increase as distance between them is decreased.

While Consulting Engineer in the U. S. Reclamation Service and adviser to the Secretary of the Interior, the writer reached the conclusion that it would be hopeless to endeavor to carry out the proposed scheme of building levees along the Gila River just above Yuma. Levees along the banks of the Gila cannot be maintained short of an expenditure for bank revetment which the value of all the land to be there protected would not justify. The situation is somewhat more favorable on the Colorado, which, owing to a lighter gradient, is less erratic than the Gila, but there, also, the fact should be recognized that it presents no ordinary problem.

The land to be protected on the Lower Colorado is of such extent and of such fertility and potential value that there should be no hesitancy in taking every step that may be necessary to bring the Colorado under permanent control.

The Secretary of the Interior was advised by the writer in 1907 that the Colorado should be given permanent banks from Pilot Knob about as far down as the Arizona boundary. He was advised that this work might cost from $50 000 to $150 000 per mile of bank, involving, as it would, permanent loose rock revetment of banks; and that the United States should be prepared for an expenditure of from $4 000 000 to $5 000 000, of which a portion should be contributed by Mexico because the benefits of the work would accrue to Mexico as well as to the United States. He was also advised that there should be created at once an International Engineering Commission, with full power to undertake the work of giving the river a permanent direct channel, as indicated, and to give consideration to all other problems arising on this stream. At the same time, the irrigation problems were reviewed, and the need of an early action for the protection of the Imperial Valley was pointed out.

At that time the Imperial Valley had to depend on a foreign corporation, *La Sociedad de Yrrigacion y Terrenos de la Baja California,* for protection against overflow and against complete and permanent submersion of the valley by the Colorado. It had to depend on

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this company, and it is still dependent thereon, for its irrigation water, which, after being diverted from the river in the United States, is turned over to the Mexican Company by which it is re-delivered into California. Such an arrangement needs the sanction and protection of higher authority. The arrangement cannot be regarded as entirely satisfactory until, under some treaty or other convention with Mexico, the right of a diversion of Colorado River water through Mexico is permanently assured to the people of the United States, and the two countries acting in harmony adopt adequate measures for keeping the Colorado out of Salton Basin.

Circumstances have favored the maintenance of the work done on both sides of the Colorado prior to 1907 below Pilot Knob. In that year the river carried more than the ordinary quantity of water; but the delta had been dry for two seasons, the river banks were free of grass and weeds, which had been largely burned off, and the over-bank flow, therefore, was far in excess of what would have occurred if the river had not been out of its natural course from 1905 to 1907, and if well-watered banks had been covered with dense vegetation. Due to its bare banks and to the consequent large over-bank flow at low elevation, the river did not rise as high and there was not as much water against the levees as there might otherwise have been. Then, too, in the spring of 1907, the cutting off of Nigger Bend and the shortening of the channel at that point by 2 or 3 miles had some effect in lowering the high water as far up stream as Yuma, and then—in 1909—came the relief at the Abejas. It may be assumed, therefore, that the behavior of the levees on both sides of the river, in 1907 and in the following seasons, is not a conclusive test of similar structures.

The writer inspected the levees on both sides of the river at high water in 1907. On the west side the levee below the dams at the Lower Mexican Heading had not been wet. At and near these two dams, completed a few months before by Mr. Cory, where the borrow-pits were on the land side, there was some seepage under the levees. Miniature, pin-head, under-water volcanoes made this apparent to the eye. The water in the borrow-pits was but little lower than in the river, consequently this seepage was caused by a pressure head of only a few feet. No doubt, it would have increased somewhat with a rise in the river. It was the result of the slow movement of the soil-water inland from the river.

On the Arizona side the borrow-pits were carrying a small quantity of water; they were not bank full. The abatis work across the borrow-pits appeared to be catching considerable trash, and each barrier of this kind checked in some measure the velocity of flow for a short distance up stream, but gave the impression that under ordinary conditions of bank submersion such structures would have been quickly cut out by the water passing around their ends. The writer was not

impressed with their efficiency, although they undoubtedly did some good under the conditions which prevailed in 1907. Mr. Grunsky.

In 1910 the writer again had an opportunity to inspect the levee on the west side of the Colorado, and noted that where the checker-board arrangement of borrow-pits had been adopted the spill from one into the next, even with very slight river bank submersion, had caused some channel cutting, making clearly apparent an inherent weakness of this arrangement.

After a number of favorable seasons these outside borrow-pits will to a large extent be refilled with sediment from the river. If located close to the bank and on no greater slope than the river, the refilling of such pits along such a river as the Colorado may occur quite rapidly. Until thus refilled and overgrown with vegetation, they are, to almost the same extent as a continuous borrow-pit, a menace to the integrity of the levee whenever the latter follows an alignment along which there is an excessive fall, as is the case on the Colorado with the Paredones levee and with those above and below the Abejas.

In reporting to the Secretary of the Interior on the matter of river treatment, the desirability of using hydraulic dredges in the construction of the levees was referred to by the writer, and he still believes that, wherever the location will permit, the bulk of the levee work on the Colorado should be done by this method. The first work would then be the excavation of an ample trench, the material from which would form the river side restraining embankment for the hydraulic fill. The hydraulic fill would then be deposited in the rear of this, refilling the trench and bringing up to the desired height an embankment so wide of base that underflow through cracked soil and through burrows would be made impossible.

The writer is of the opinion that the control of the Colorado between permanently fixed banks should not be projected beyond some agreed point near the Arizona boundary line. If the river is kept on a proper alignment down to that point, it can wander about over a large lower portion of the delta without menace to the Imperial Valley. It will then, as it does to-day, send its waters, at least in large part, into the Volcano Lake region, and will warp up first one and then another section of its lower delta region. In the far future, then, the time may come when complete control to the Gulf can be economically justified.

If controlled mainly by giving the river a direct alignment between permanent, well-protected banks, supplemented with levees, as far down stream as here indicated, the protection of the Imperial Valley and the whole Alamo and New River country will be complete, if the cross-levee to the northward of Volcano Lake from the base of the Cocopah Mountains to the Colorado at the Lower Mexican Heading

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and up the river to Pilot Knob be brought up to proper height and be protected adequately, particularly against wave action.

The writer desires to note that the various estimates of flow of the Colorado, even those based on gaugings, are to be accepted with caution. Some additional information relating to methods of gauging should be given before these estimates are accepted as conclusive. This applies particularly to the measurements made at the river's high stages. It is understood that some special precautions are now being taken to insure accuracy of soundings and the correct placing of the current meter at the intended depth; but in 1907 the writer knows that this was not done, and that the maxima then recorded are too great by from 15 to 20 per cent.

The two-point method of gauging was at that time used. The gauging was made from a car supported by a cable. In mid-stream the car was from 5 to 8 ft. above the water surface. A sounding was made immediately before the velocity was measured. The heavy sounding lead was lowered from the car to the water surface and a reading of the paid-out line was taken. Then it was lowered to the bottom of the river and another reading was taken, the difference between the two being recorded as the depth. No allowance was made for the line swinging out of the vertical, due to the force of the current, which was about 7 ft. per sec. in mid-stream. By observation from shore on June 24th, 1907, the correction to be applied to the recorded mid-stream depth of 45 ft. was found to be about 6 ft. The recorded depth determined the required immersion of the current meter. When the meter was supposed to be at one-fifth depth, or 9 ft. below the surface, it was in reality barely below the surface; and when it was supposed to be at four-fifths depth, it was but little deeper. The only value of the velocity measurements as made lies in the fact that they can be used to determine surface velocity. They were recorded, however, as applying to the two-fifths and four-fifths points, with a resulting over-estimate of mean velocity. In this particular case the recorded discharge should be reduced 15% to give approximately the flow of the river on that day.

The errors of observation as above noted were, of course, called to the attention of the Reclamation Service and the United States Geological Survey, but no correction has been applied, and the writer does not know how long such methods of gauging were maintained at that station.

In Water Supply Paper No. 249 of the U. S. Geological Survey, page 45, the discharge is noted for June 24th, 1907, at 112 000 sec-ft. Of this quantity 11 000 sec-ft. were flowing in the north channel of the river and 101 000 sec-ft. were recorded for the main stream. According to a proper interpretation of the measurements on that day, the main stream carried only 86 000 sec-ft., and probably somewhat

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less. The aggregate discharge for that day should have been noted at 97 000 sec-ft., instead of 112 000 sec-ft. The error in the main stream gauging was about 17%, or the recorded main stream flow should be reduced 15 per cent. The same correction should undoubtedly apply to the entire period in 1907 during which the river was at or near its highest stage. The writer has no information as to whether the same method of gauging was still in use in 1909. If it was, there can be no doubt that in that year, also, there must have been a considerable over-estimate of discharge.

Mr. Cory is right in his conclusion that the construction of reservoirs within the water-shed of the Colorado will have some beneficial effect in equalizing the stream flow and in reducing the maximum discharge. It is immaterial whether or not the storage possibilities have been over-estimated. This can only affect the extent of the reservoir influence; and yet, whatever this may be, it will not change the river problem, except as to its magnitude. The river will still have its varying volume of flow. Its channel will adjust itself to the new conditions. The same lack of channel capacity will be felt at high stages on the lower river as is felt to-day, and the same problem of keeping the delta channels of the river on the Gulf slope will confront the river engineers of the future as confronts those of to-day, except only that there will be somewhat less water and correspondingly less silt to be reckoned with.

Perhaps a word should be said about the Laguna Weir. The change in the original design from a paving with large blocks of rock to a paving with concrete was due to the fact that the granite which had to be removed at each end for the sluice-ways was not of a character which could be used for this paving work. It was anticipated by the engineers who designed and approved the structure that the granite at the "Lagunas" which appeared to be disintegrated and rotten at the exposed surface would be of good quality when cut into; but, as the work progressed, it was found that the hills afforded very little sound rock within the limits of the prescribed excavation, and that at the best quarry sites near the weir only a small proportion of the rock was suitable for paving. This situation was carefully considered before a modification of plans was recommended by the Board of Consulting Engineers which had this matter under consideration in 1906.

The writer, though not desiring to criticize unfavorably the works which have been carried out at Laguna for the diversion of water into two canals, one on each side of the river, has never been in full accord with the general arrangement of the structures at that point. The plans for these structures were approved before he had anything to do with the Reclamation Service, and no one is more pleased to learn that they are rendering satisfactory service. Nevertheless, he

Mr. Grunsky. would call attention to the fact that the decrease of depth of overflow over such a structure, resulting from its great crest length, 4 700 ft., is not necessarily of sufficient importance to outweigh the disadvantages that will result from the use of a dam with so long an overfall crest. The storage of silt in the reservoir space above the dam, in the case of such a river as the Colorado, is of no importance. This space, except for a channel of ordinary width leading to the sluice-gates, is soon filled with silt and overgrown with willows and cottonwoods. There is a strong tendency for drift to lodge, and, within a few years, there may be such obstruction to a concentrated flow in lines parallel to the original course of the river that it may be difficult to keep the waterways open to the two ends of the weir without permitting one or the other to parallel and endanger the structure. The conditions at the weir, in a few years, may be such that water will go over it at uneven depth, and, in that event, the great length will fail to accomplish fully the desired purpose. At any rate, there will be trouble from time to time in keeping the water on a proper course to the two ends of the dam. The removal of silt which the sluice-gates will permit is dependent on the depth of the sluice-ways below the sills of the canals. The silt which the river will deposit in the large channel leading to a sluice-way, while the water is forced to rise to the crest of the dam, can of course be sluiced out from time to time by opening the gates, but this will be restricted to the deposit in a channel. It is not the accumulation in a reservoir, and this fact should not be lost sight of in planning such structures. Even this, however, may be worth while. The writer believes that the river should have been forced into a definite position above the weir by making a section thereof, probably 800 ft., a few feet lower than the rest of the structure and then arranging flanking walls to be overtopped at highest stages of the river and to serve as training walls of the large-capacity channels leading to the sluice-gates, which would, as at present, serve to keep the channel open to the head-gates. One great advantage of such an arrangement would be the concentration and depth of flow necessary to carry drift of whatever size over the dam.

Mr. Clarke. CLARENCE K. CLARKE, ESQ.* (by letter).—Although this paper and Mr. Sellew's discussion of some phases of the subject seem to have covered the ground quite thoroughly, it appears to the writer that room remains for a few statements of fact concerning the "rough and ready methods" of dealing with the river which Mr. Sellew does not accept as a solution of the engineering problems pertaining to the subject.

Being one of the men referred to by Mr. Sellew as "red-blooded fighters, who did not know when they were whipped, but * * *

* Chf. Engr., Palo Verde Mutual Water Co.

fought on to victory unmindful of its cost," the writer desires to correct an erroneous impression. The crevasse in the river bank and levee in 1907 was closed by railroad men, who used methods with which they were familiar; they tried no experiments and devised no new engineering practice. They knew they were not whipped at any stage of the game, and they kept at the job because they had done the same thing on a smaller scale many times and were confident of ultimate success. Mr. Clarke.

The use of the water of the Colorado for the irrigation of land creates the problem of control. There is no other reason for subjecting that river to control. The problem, so-called, as the writer sees it, presents three phases: 1, direction of the flow of the river and control of its meanders, involving bank protection; 2, exclusion of overflow water, by levees, from land susceptible of reclamation and irrigation; 3, safe and certain diversion of water from the river into irrigation canals.

The Lower Colorado has no fixed channel, because of the character of the soil, which is a deposit of silt, readily eroded. The current swings back and forth, cutting the banks and changing the meander line, and an apparently insignificant obstruction sometimes causes shifting of the channel for miles. A sudden shift of the channel from one bank to the other may leave a canal intake dry and put an irrigation system out of business. It may cut across a tongue of land, changing a wide bend into a tangent and forming a new bend lower down. Therefore, one factor of the problem is the maintenance of a defined course of the river between permanent banks. That may be accomplished by adequate protection of the banks and control of the meanders. It is advisable and feasible to straighten the course of the river at some points, and by increasing its velocity, to check its tendency to swing from side to side. The shortening of the river, resulting from the elimination of many horse-shoe bends, would increase the scouring action of floods and their silt-carrying capacity, and greater volumes of sand and silt would be transported to the lower delta and deposited there, building up large areas of land and accelerating the work that the river has been doing for ages. By the control of the lower river, the floods could be diverted into the basin of the Laguna Salada, and that shallow basin could be converted by silt deposit into an arable and fertile valley. In time, of course, the deposition of silt would work back, and the normal grade of the river would be re-established, necessitating the adaptation of levees to the new condition.

It is the writer's purpose to point out that this is not a problem calling for the invention of new methods of river control.

Bank Revetment.—Effective bank protection can be provided most readily and economically by laying a railroad track on the top of a

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levee; and, as other writers have suggested, it may be applied in sections as needed. That tracks on the levees along the Lower Colorado are indispensable to their maintenance and the defence of the banks against undercutting attacks by the river, has been the writer's fixed conviction since his first experience with the river, and he notes with gratification that Mr. Sellew has revised his former opinion on the subject and concluded that the railroad track facilitates the work of levee protection to a degree that more than offsets the cheapness of the methods heretofore used by the Reclamation Service on the Colorado.

Having the "firing line" established by the track on the levee, it is important to provide an abundant supply of "ammunition" and means of rushing it to "the front." Quarries and gravel pits are opened, and are equipped with derricks to handle large rock, and steam shovels to handle small rock and gravel. When the river begins bank cutting and threatents to undercut the levee, train loads of gravel and rock are taken to the point of attack and dumped at the toe of the levee. Gravel is dumped first in order that it may underlie the heavy rock and prevent the sluicing out of silt and sand where the rock lies on the bottom. When the bank caves, the rock on the berm falls into the trench dredged for it by the river, and if the scouring continues and the trench is deepened, more rock is dumped to reinforce the revetment and raise it to the crest of the levee. Small rock and steam-shovel material serve to fill and face the revetment. When such revetment has dropped to the river's lower scour plane and has been completed by proper reinforcement, the work of bank protection at that point is done permanently, and the levee thenceforth is safe from undercutting.

An illustration of this method of bank revetment was the saving of a section of the C. D. Co. levee, as a part of the work of closing the break with what Mr. Cory calls the Clarke Dam, in January, 1907. The river swung in against the levee north of the break and threatened to destroy 1 400 ft. of the embankment and widen the crevasse from 2 600 to 4 000 ft. The width of the berm between the toe of the levee and the bank varied from 30 to 50 ft. for the greater part of the 1 400 ft., but near the north end of the section the current had washed away most of the berm, and in the bight the water was close to the toe of the levee. To divert the current from this bight, a jetty 90 ft. long was built out from the upper side. The jetty consisted of two rows of piling, 5 ft. between rows and 8 ft. between piles in the row, the piles being "staggered" or alternated. The piles were 40 ft. long, and were driven in 26 ft. of water. Before the completion of the jetty, the discharge of the river was increased to 40 000 sec-ft. by a freshet in the Gila. Gila freshets carry great volumes of heavy silt in suspension, and this material drops readily when the velocity of the current is checked. It was not necessary to oppose any other barrier or baffle

than the piles to this heavily laden flood, and no brush was placed in the jetty. Eight days after the completion of the jetty, the river discharge fell to 15 000 sec-ft., and a bar was exposed on each side of the jetty and extending 13½ ft. beyond its outer end, where the water had been 26 ft. deep. The rapid formation of the bar was due to the Gila silt. To accomplish similar results with Colorado River water alone, more obstruction, in the form of fascines or wire entanglements, would be required, and the process of bar formation would be slower. Mr. Clarke.

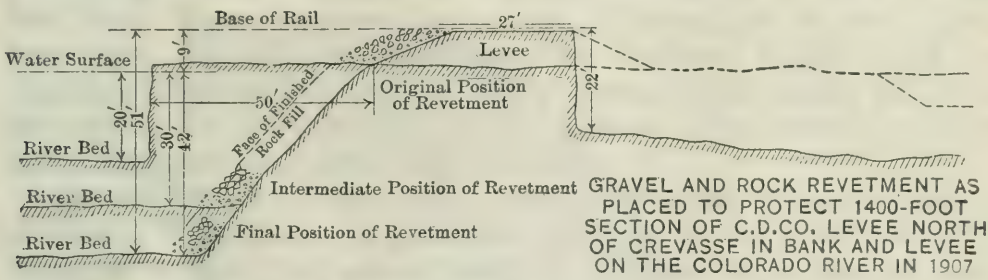
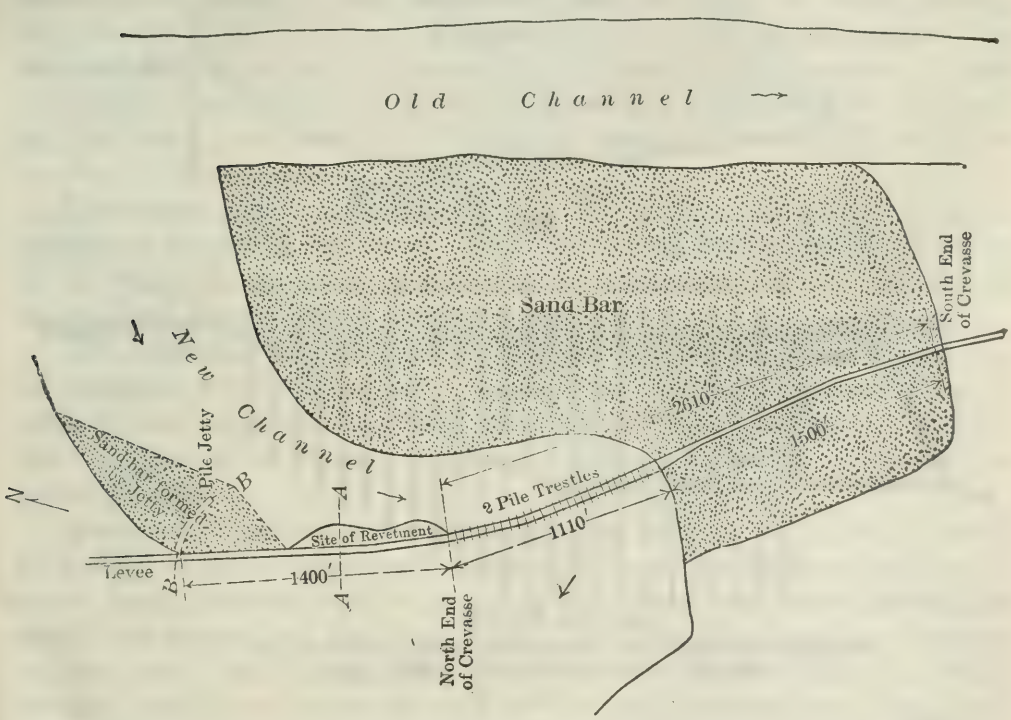


FIG. 84.



PLAN SHOWING METHOD OF DEFENDING LEVEE AGAINST ATTACK BY RIVER DURING CLOSURE OF COLORADO RIVER CREVASSE BY S.P.R.R. ENGINEERS IN 1907

FIG. 85.

When the jetty was built, the levee embankment was widened toward the river with gravel and rock, the track was moved to the extended fill, and large quantities of gravel and rock were unloaded, covering the berm to a width of from 8 to 10 ft. from the toe of the

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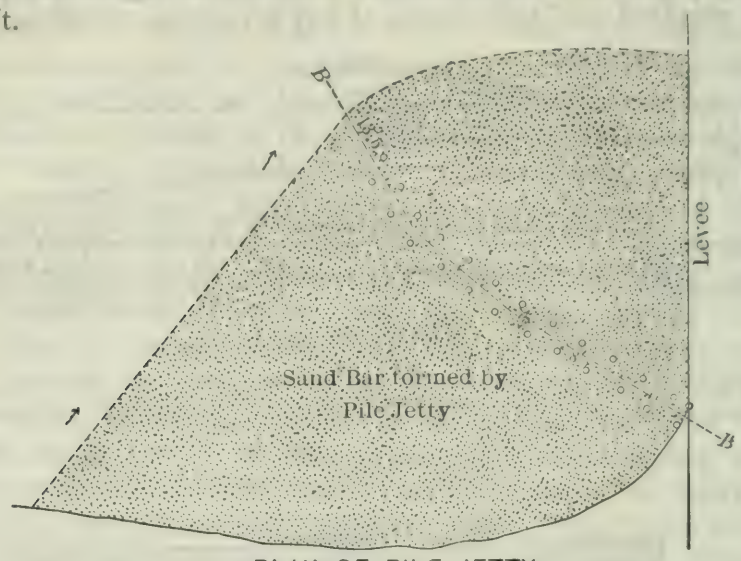
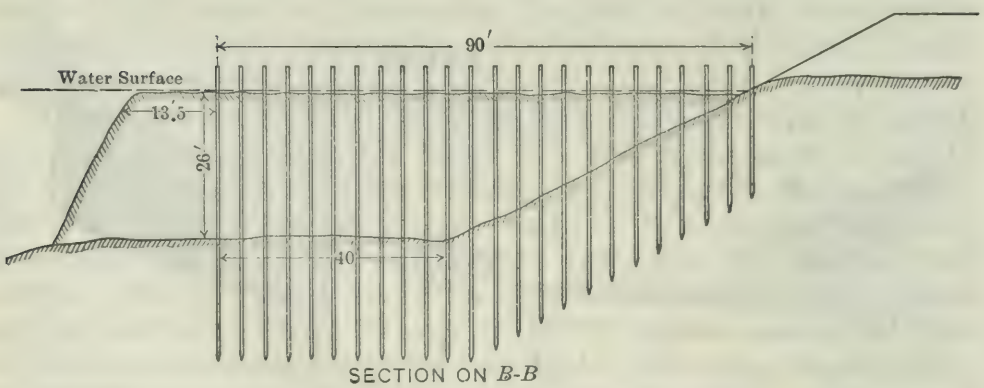
levee. As the river cut away the bank and deposited the gravel and rock at the bottom, where the material was needed, more rock was unloaded. The depth of water at the bank, when the cutting began, was from 20 to 25 ft. This depth increased to 42 ft. as the river dredged a trench, and at that depth the scouring action ceased. When the revetment was completed and the 1 400-ft. section of levee had been protected, the height from river bottom to base of rail on the levee was 51 ft.



PLAN OF PILE JETTY

FIG. 86.



SECTION ON *B-B*

SECTION OF SAND-BAR FORMED BY PILE JETTY FROM C.D.LEVEE ON THE COLORADO RIVER, 1907

FIG. 87.

Gravel is a factor of vital importance in this method of placing rock on a sand or silt bottom, a detail which appears to have escaped the notice of some of the "innocent bystanders," who watched the closure of the crevasse and predicted utter failure of the work. The plan of providing revetment material at a threatened point and leaving to the river the work of dredging a trench and placing the rock, has been approved recently in a practical way by so eminent a river engi-

neer as Mr. Sellew, who used the method last year in defending one of his levees from attack. As he does not refer to the use of gravel as a bed for rock revetment, presumably that essential detail of the method was omitted. Mr. Clarke.

This method of bank protection, however, is no innovation, and, in his application to the closure of the Colorado crevasse of practice that is old to "firing-line" railroad engineers in the West, the writer cannot lay claim to originality. He used the same method on the San Pedro River, between Benson and Fairbank, in Arizona, many years ago. In 1891, the roadbed of the Northern Pacific along the Puyallup River, in Washington, was protected in identically the same way within less than 4 miles of the Tacoma depot, where doubtless many "innocent bystanders" observed the work.

Control of the River.—The general application of this system of revetment to the banks of the Lower Colorado, in the writer's opinion, would solve, simply and at comparatively low cost, the problem of keeping the river within defined bounds and protecting levees against undercutting between flood seasons. Combined with the straightening and shortening of the river's course and control of the meanders, this revetment of banks would increase the efficiency of the channel. Trees and other obstructive growth on the bars and flats would be removed by the river itself, and its carrying capacity would be sufficient eventually to take care of floods.

Experience has demonstrated that the revetment here described, consisting of gravel and rock, and placed from the lowest scour plane to the crest of the levee, constitutes a permanent barrier and solves the problem of control of the Colorado.

The resulting rock-reveted permanent barrier would be similar in all essential features to that contemplated in the suggested plan of river control attributed to Gen. Marshall, which involves the excavation of a trench to the ascertained scour line by the use of dredges or power shovels.

A serious objection to the suggested plan referred to is that it would be applied without reference to immediate need for defense, and would entail large expenditure for dredging and revetment where protection might not be required for many years. The cost of excavating trenches to the scour line, along both banks of the Lower Colorado, with dredges, would be enormous, and it has been demonstrated that such expenditure is quite unnecessary. The river can be depended on to do all the dredging and to place the gravel and rock at the bottom of the trench.

Only the National Government could or should undertake the task of controlling the Colorado, as it has prosecuted the work of protecting lands along the Mississippi against inundation. The first step in the project obviously would be to construct levees along both banks

Mr. Clarke. and lay tracks thereon. During the progress of this work, gravel and rock quarries should be opened and equipped with the requisite loading appliances, and cars should be provided to carry the material to the front. In case of attack on the levees by the river, material would be sent to the threatened points, and the work of permanent revetment would be done at those points. After the completion of the levees, the means and material for revetment would be kept in readiness; during each flood season the river would be observed vigilantly; and every attack on the levees would be met promptly.

This method would distribute the work over many years. Conceivably, 50 years or more might pass before the banks would be reveted throughout the course of the lower river, but during that time the stream would be virtually under control. The cost also would be distributed, and no very great expenditure probably would be called for in any one season. The ultimate result would be absolute control of the river between permanent barriers defining its course.

Control of Meanders.—Emergency methods of control of meanders and of the direction of the channel, used on the Colorado, are simple and comparatively inexpensive, and may be applied by settlers to the temporary protection of their riparian lands. The writer had occasion to resort to these methods in January, 1913, for preservation of the flow of water through the intake of the Palo Verde Valley irrigation system.

The Palo Verde intake is cut through a rock projection into the lower part of a wide bend in the river. The river developed a tendency to cut across the bend by the chord of the meander arc, and, if unchecked, the channel would shift toward the Arizona shore, and leave a wide bar in front of the intake. This tendency was shown in the cutting away of the nose of a bar that extended from the Arizona or eastern shore to about the middle of the bend, and it was necessary that the bar should be maintained. This was accomplished by driving posts and placing fascines and sand bags on the point of the bar. A fascine jetty was built out from the Arizona bank, 1½ miles above the bar, and pointing down stream at an angle which would direct the current into the bend above the intake. The cost of this work was small, and the effect was all that was required. The channel was kept in the bend close to shore, and the supply of water through the intake was assured for the season.

By similar means, when the river begins cutting the bank and threatening destruction to lands or levees, a bank-caving current may be converted readily into a silt-depositing eddy. Fascine jetties—rows of driven posts with bundles of brush placed between them and anchored with sand bags—may be built quickly by the farmers, and danger may be averted if detected early and if the remedy be applied at the first indication of trouble. Such temporary defences are avail-

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able in advance of the permanent system of protection, consisting of levees, railroad tracks, and rock revetment, and may be built by intelligent farmers without the aid of engineers.

Levee Construction.—The writer did not participate in the exhaustive discussion of the relative merits of river-side and land-side borrow-pits in levee construction on the Colorado, which followed the failure of Col. Ockerson's levees and was carried on recently by many engineers in the technical press. The subject having been reopened by Mr. Cory and Mr. Sellew, however, it seems pertinent to call attention to a few facts which have been overlooked or ignored by both.

Mr. Cory explains clearly and correctly the failure of the levee below the Hind Dam in December, 1906, which was due solely to the absence of a muck-ditch under the levee. He gives reasons for the omission of the muck-ditch, chief of which was the supposed necessity for confining expenditure, as closely as possible, to the repair of the break. Mr. Cory says the desirability of muck-ditching was fully realized, and it was a part of the original design, but because levees had been built without muck-ditches on the Colorado, and some one advanced the theory that a cracked adobe foundation would be made tight by pressure when wet, the vital necessity of muck-ditching was not recognized, and the builders of the levee decided to "take a chance." The theory of the closing of adobe cracks proved unsound. Water followed the cracks under the levee and caused the break.

Mr. Sellew admits that water did go under the levee because of the absence of a muck-ditch, but he contends that "the damage caused by a head of 15 in. against the levee was due primarily to the presence of land-side borrow-pits."

The borrow-pits, 3 ft. deep, were separated from the levee by a berm 40 ft. wide, and it is manifestly absurd to assert that there could be any relation between the pits and a 15-in. head against the levee. The pressure angle of the greatest head against any levee on the river would not reach the pit. According to Mr. Sellew's theory, an inside borrow-pit ½ mile back of the levee would increase the static head and cause failure of the work. Mr. Cory left an opening for attack when he said the increased total head was "the only pertinent objection to land-side borrow-pits." Experience has demonstrated, however, that the objection is impertinent, irrelevant, and immaterial. No land-side borrow-pit levee, properly muck-ditched, on the Colorado, has been broken or even damaged by the river, but every river-side borrow-pit levee has been breached or destroyed, and every flood season since levees were first built on the river has seen a strenuous fight to maintain them. The river-side borrow-pits are invitations to the river to side-wipe the levees with currents of eroding velocity, and a levee not blanketed heavily with gravel, or rip-rapped with rock or mattresses, melts away like a bank of salt. Every levee breach has been

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caused by current erosion, not by static head or pressure. No levee has been pushed over by the river.

"Last Word" on Borrow-Pits.—Quoting from Mr. Sellew's discussion, page 1482:

"Regarding the advantages and disadvantages of land-side and river-side borrow-pits, the writer has fully expressed himself, so that it is not necessary to comment further. Particularly is this so when the author [Mr. Cory] states that he 'realized that this [land-side borrow-pits] was not in accordance with the usual practice.'"

The implication that Mr. Cory confessed judgment against the land-side borrow-pit and yielded every point to Mr. Sellew in the argument, seems to the writer no more than a verbal quibble. A practice may be "usual" and still not the best possible under all conditions.

Doubtless there is satisfaction in the belief that one has routed all opponents in argument and said the last word on a debated subject, but it does not appear to the writer that any one of the eminent engineers engaged in the borrow-pit discussion has warrant for "laying that flattering unction to his soul."

It might be inferred from Mr. Cory's statement, that the land-side borrow-pit method of constructing the C. D. Co. levees in 1906-07 was "decided on after careful consideration of the advantages and disadvantages," that there was doubt on the subject in the minds of the men on the job. The matter may have been—in fact was—debated persistently by the "innocent bystanders," but there never was any question of methods in the minds of the railroad "firing-line" engineers having direct charge of construction. It is and has been for years, not only the usual practice, but the invariable rule of railroad engineers in the West, that nothing between an embankment and a river shall be disturbed. The protection afforded by earth in place and by trees and brush must be maintained intact. That was why the land-side borrow-pit system of construction was adopted as a matter of course. It was followed until the bystanders, tired perhaps of being innocent, got themselves appointed as a "consulting board" and after two attempts succeeded in getting an imperative order from financial headquarters to the engineers on the job to conform to the usual practice of the levee builders on the Yuma Project. Had they succeeded earlier, the levee could not have been built at all during the flood period. From the moment the water came over the river bank, work of any sort on the river side of the levees was impossible.

Experience in railroad construction and maintenance in regions subject to floods, in the building of dams and levees in the Delta of the Colorado and in the Palo Verde Valley, and in the control and maintenance of irrigation canals of large capacity, has left the writer in no doubt concerning the essential principles of levee construction on the Lower Colorado River.

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The first requisite pertaining to the location of a levee in this region is a careful underground survey to determine the sort of material to be encountered in excavation for muck-ditches. Large deposits of unfit material, such as adobe, heavily alkaline soil, and certain kinds of sand, must be avoided by detours. Seepage is the only factor detrimental to the land-side borrow-pit plan, and that is to be eliminated by the muck-ditch. The theoretical additional head against the levee, due to a borrow-pit 40 ft. in the rear of the levee, is a negligible factor. Under conditions actually encountered on the Colorado, there is not any additional head, either in fact or in theory. That objection to the land-side borrow-pit is not even theoretically plausible.

Essential Features.—The essential features of efficient levee construction on the Lower Colorado are: location avoiding bad ground and careful muck-ditching to exclude seepage; land-side borrow-pits, leaving all natural protection on the river side undisturbed; gravel blanket to resist erosion and to minimize attacks by burrowing animals; railroad tracks on top of the levee to facilitate maintenance and to make possible the protection of banks and levees by permanent rock revetment.

Mr. Sellew is unquestionably right in his conclusions:

"That in their present state, the levees are at the mercy of the meanders of the stream, except throughout those portions where a railroad occupies the levee top"; and, "That bank revetment, permanent in character and reasonable in cost, when compared with other methods, can be readily and quickly accomplished with rock placed from railroad trains operated on the levee tops and connecting with existing quarries which are within a reasonable distance."

The first conclusion is sustained by the history of all the Colorado levees, and the correctness of the second was demonstrated clearly in 1907 by the holding of the 1 400-ft. section of levee against the attack by a flood of 40 000 sec-ft. at the time of the last closure. The permanency of rock revetment thus constructed, however, depends on the placing of gravel as a bed for the rock. That is a vital detail of the method.

In rip-rapping levees and reveting the banks at the toes, in the opinion of the writer, grading to uniform slope is not only unnecessary, but inadvisable. It is better to place the gravel and rock on the natural ground. Setting rip-rap by hand on the face of a levee makes a neat looking job, but it is needlessly expensive, and the protection is less effective than that secured by the gravel and rock method.

Before leaving, finally, the subject of closure of the crevasses and the preservation of Imperial Valley in 1906-07, the writer desires to put on record the fact that the accomplishment of the work was due primarily and exclusively to the independent judgment and courage

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of Mr. Harriman, who persisted in his belief that the breaks could be closed, and his determination to close them in the face of opposition, and regardless of the positive assertions of a host of eminent engineers that closure was a physical impossibility.

Submerged Weir.—Mr. Cory has described briefly the submerged rock weir placed across the river at the intake of the Imperial Valley Canal in the summer of 1910, and has mentioned its unexpected stability and durability. Mr. Sellew does not discuss the weir or its effects directly, possibly because he did not view its construction with favor, but his comment on Mr. Cory's statement concerning the permanency and efficacy of rock fill in the crevasses has bearing on the subject. Of the rock fill Mr. Sellew says:

"How it would act as an overflow weir, carrying 150 000 sec-ft., even if covered with concrete, is problematical. In addition to a concrete top, crest and foot-walls appear to be necessary to prevent the top from being undermined, and, if these are provided, the resulting structure will not differ materially from Laguna Dam either in design or cost."

This is a case in which accomplished facts fail to justify speculative theory, and it appears to be in order for the writer, as the builder of the weir, to state the facts and show the actual effects of the installation.

Construction of the weir was necessary because of the recession of the bed of the river, following the diversion of all its flow into the Abejas channel, and because of the lack of a powerful dredge to keep the floor of the 1 900-ft. intake from the river to Hanlon gate as low as the sill of the gate. The discharge of the river was diminishing, and danger of complete diversion of the channel from the intake was imminent. The placing of a weir to check recession and direct the current was the only practicable rapid method of returning the water to the canal and saving a $6 000 000 crop in Imperial Valley.

A trestle, 960 ft. long, composed of four-pile bents, 15 ft. between centers, was built across the river, below the intake, to the Arizona shore, at an angle of approximately 70°, the Arizona end being the farther up stream. Caps, stringers, ties, and rails were placed on the structure, and steam-shovel rock from the adjoining quarry was dumped from the trestle and spread to a width of 50 ft. No brush at all was used, and no derrick rock was dumped. As the weir was designed to be temporary, no gravel was placed either before or with the rock. The discharge of the river during the progress of the work was about 9 000 sec-ft.

The weir was constructed in 21 days, at a cost of $22,500, of which amount $4 000 was recovered in salvage of rails, ties, stringers, and caps. The quantity of rock used was 12 800 cu. yd., of which 9 600 cu. yd. were placed in the weir, the rest being used to revet the banks

and raise a spur track above the high-water mark of 1909. The angle of the weir to the current served to point the channel to the California shore, and also served other purposes familiar to "firing-line" railroad engineers. Mr. Clarke.

The weir arrested the recession of the river bed, turned the water into the intake, averted a water shortage in the Imperial irrigation system, and saved to the farmers of Imperial Valley crops worth $6 000 000 or $7 000 000. Also, the successful construction and operation of the weir demonstrated that the farms of Yuma Valley, dried out and virtually abandoned for 5 or 6 years, could have been watered at small expense, and could have produced and marketed crops to the value of $1 000 000 a year while awaiting the completion of those great engineering monuments, the Laguna Dam and the Colorado River Siphon. It is evident that the lands of Yuma, Cibola, and Parker Valleys can be provided with water by the construction of weirs, and brought under cultivation at nominal expense.

Although the piling of the trestle was cut off above the rock fill and removed in the spring of 1911, the rock weir remained in place. How such a fill would stand an overpour of 150 000 sec-ft., even without a concrete top, is not problematical in any sense. The weir at the intake has endured two freshets, one of 150 000 sec-ft. and the largest on record, and it still remains. It has neither crest, core, nor footwall of concrete, and it is not covered with concrete, but its top has not been undermined. It differs radically and widely from Laguna Dam in design and cost, but its function is the same. A permanent rock-fill weir, built with gravel, large rock, and steam-shovel rock, and having a cross-section width of 200 ft. on the bottom, certainly would stand any overpour that the Colorado could muster, and its cost would be comparatively small.

Useless Dredging.—Although the work of a dredge in the intake above the concrete gate was of material aid in getting an adequate supply of water to the canal, the writer cannot concur in the opinion that the operation of the dredge *Imperial* below the gate was of the slightest efficacy or value at any time. The dredge has dug a hole below the gate; the hole has been filled with sand; the dredge has dug it again, and so on in one continuous round of costly futility, like the labor of Sisyphus. Had the *Imperial* been placed above the gate, it would have been useful. When the banks of the Alamo channel, constituting the main canal, had been raised and strengthened to carry safely a large flow, the discharge of from 2 500 to 3 000 sec-ft. through the gate solved the problem of water supply for the valley canal system. In an hour and a half 2 500 ft. of water sluiced sand from the canal below the gate to the depth of 1 ft. The Alamo Canal is an efficient channel for 5 miles below the gate, when carrying 2 000 sec-ft. or more, and dredging is wholly useless.

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When the writer took charge of work on the C. D. Co. system, he found two large dredges, the *Delta* and *Gamma,* at work in the main canal at an operating cost of $9 000 a month. The dredges were useful in straightening the course of the Alamo, cutting across bends and building up banks, but when they operated in the canal to remove silt and sand, they only moved a hole up and down stream. Careful instrumental observations and cross-section surveys showed that the floor of the canal behind a dredge was at the same level as before the dredge reached it.

The writer put the dredges out of commission and tied them up, thereby saving $9 000 a month. It is doubtful if a dredging barge, occupying more than one-third of the width of a canal carrying Colorado River water (laden heavily with silt and sand), can be operated efficiently to remove deposits.

The removal of quicksand from the Imperial canals is a problem to be solved by the managers of the main supply system. Sand in the canals of the Mutual Companies constitutes a menace, and is a cause of great increase in maintenance expense. The deposit of sand in these canals and in farmers' laterals is avoidable. Doubtless a small percentage of sand brought through the head-gate is removed by the *Imperial,* but all of it could be removed more economically by other means at the lower end of the Alamo main, and it is obviously the business of the C. D. Co.'s Receiver to take care of that sand instead of sluicing it out upon the valley farms.

Summary.—The writer has endeavored to make clear, by statement of fact and description of work accomplished, that there is no abstruse problem involved in placing the Lower Colorado under control and confining its floods to a determined course.

The meanders of the river are subject to direction by simple and inexpensive means, the current at the early stage of its wandering tendency responding as readily to the guiding hand as does a bridle-wise horse to the touch of rein upon the neck.

The permanent revetment of the banks has been demonstrated to be comparatively easy of accomplishment by methods familiar to, and applied by, railroad engineers in the West for many years.

That a railroad on the top of a levee is indispensable to maintenance and to application of the rock-fill method of bank revetment has been shown conclusively by experience. Costly experience has proved the unwisdom of removing any natural protection from the berm on the river-side of a levee, and has established the correctness of the railroad "firing-line" practice in the construction of embankments along streams, particularly those flowing through regions of alluvial formation.

Submerged weirs, to raise the bed of the river, arrest the recession of scour planes, and divert water with certainty and safety into canal

intakes, may be built as permanent structures by the simple and inexpensive method of dumping gravel and rock from trestles, without placing any crest or core-walls, or using a pound of cement. Mr. Clarke.

The operation of dredges for the removal of sand from main irrigation canals is ineffective and a waste of money where water can be carried in large volume at sluicing velocities.

U. S. Marshall, Assoc. M. Am. Soc. C. E. (by letter).—The author's statements regarding the very unfavorable soil report on the Imperial Valley, issued by the U. S. Department of Agriculture (which actual results have proven to be unjustified), and the belief expressed that, had this soil report been issued before the reclamation of the valley was well under way, the effect would have been a complete condemnation of the project, are serious matters and worthy of most attentive consideration. The writer has had similar experience, and has no doubt that many others interested in reclamation work have encountered this same bugaboo of an adverse Government soil report. It is certainly the duty of the U. S. Bureau of Soils to rectify some of the wrongs done to different parts of the country through adverse reports on various soil types. The writer knows of no case where this has been done in a way which has had any practical effect. Mr. Marshall.

The object of a soil survey should be to determine what the land is adapted to and what methods of cultivation are indicated and required. The findings of a survey should be, by all means, of a constructive and not of a destructive nature, and the chief purpose should be to find some beneficent use for the soil surveyed. Any report that condemns a soil type as worthless, unless based on the most thorough investigation, necessarily does a most serious and inestimable injury to the area reported on.

It should be the office of the Government to encourage the development of waste land, but certainly some of the soil reports would discourage even a preliminary investigation of the areas in question. Because of the usual high respect for Government experts and their disinterested attitude in such matters, their statements, official and otherwise, are generally taken as expressions of ultimate facts, and unimpeachable as to the conditions of the soils reported on. However, in the West, to-day, there are notable cases of great districts, with proven agricultural value, which were condemned by the Government Bureau of Soils.

The writer's experience with an adverse soil report will serve as another example, supporting these statements. While one of the engineers was making a report on a proposed 60 000-acre levee reclamation project in the Sacramento Valley, California, on which it was estimated $2 000 000 would be spent for levees and drainage works, the following developed:

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The project had advanced to the point where the parties who were to finance it had paid for 26 000 acres of the land involved, and plans were being hurried for the reclamation works, when the U. S. Department of Agriculture's "Soil Survey of the Marysville Area, California," was called to the attention of one of the several representatives of the purchaser. This report, issued in 1911, was the work of Messrs. A. T. Strahorn, W. W. Mackie, H. L. Westover, L. C. Holmes, and Cornelius Van Duyne, and on page 30 contained the following comment regarding that part of the area in which this project was located:

"When protected from overflow by levees, this soil will be saturated for some portion of each year on account of the high water level outside the levees. Extensive drainage ditches and pumping plants would remedy this to a considerable extent, but to keep down the level of the water table for such extensive areas would be very expensive, and only the most profitable crops would justify the expense. The physical condition of the soil is about as unfavorable as it possibly could be, owing to water-logged condition and consequent nonaeration. Should the type ever be thoroughly protected from inundation, the agricultural value of the section will probably depend upon the introduction of crops suitable only to low, moist land that will yield a considerable return for the labor and money invested.

"The following table gives the results of a mechanical analysis of a sample of the soil of this type [amounts given in percentage by weight]:

Fine gravel.	Coarse sand.	Medium sand.	Fine sand.	Very fine sand.	Silt.	Clay.
0.0	0.5	0.7	3.4	1.8	41.0	52.7"

This report, coming from the Government, and obviously unbiased, seemed to this representative to be a solar plexus blow to the project, and he felt that the purchaser had been misled. The parties effecting the sale felt that their reputation was in jeopardy. They had never seen the soil report, but were positive, because of actual results, that the soil was very productive. The engineers were censured for failing to bring the report to the attention of the parties interested, and it seemed certain that the whole project would fall through.

In an effort to re-establish confidence, a large number of representative farmers, experienced in handling bottom lands, were taken over the area and asked to pass judgment on the soil. All agreed that any land supporting a heavy tule growth was valuable when reclaimed, as much of this identical soil which had been reclaimed in the Sacramento Valley produces great crops. Much of the land in this area, however, is not covered with tules, and these bottomland farmers would not vouch for it because they could see no native tule growth, which was the basis for their conclusions. In this diffi-

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culty the writer had recourse to the United States Government surveys made in 1854 and 1864. In these notes frequent reference was made to tule growth encountered. On platting these, a map was obtained showing that the tule growth in 1864 covered nearly the whole of the area in question. This was accepted as convincing proof of the continuity of soil type, and the purchaser was reassured to the extent that reclamation is now going on.

There will thus be created for the community a vast acreage of exceptionally productive ground which, had it been impossible to offset the adverse report of the soil experts, would have continued a worthless expanse of duck marsh, and the reputation of many men of high standing would have been shattered and confidence in their integrity and judgment lost. Further to re-establish the reputation of the land and prove the mistaken character of the soil report in question, comparisons were made with the "Soil Survey of The Woodland Area, California," by the U. S. Department of Agriculture. This area adjoins that of Marysville, and was reported on by Messrs. C. W. Mann, J. F. Warner, H. L. Westover, and James E. Ferguson. The soil type commented on so adversely in the Marysville Area is reported on as follows in the survey of the Woodland Area, page 25:

"Where this soil type is protected from overflow and is free from alkali it is adapted to grain, sorghum, Egyptian corn, hay, and other forage crops.

"The texture of the soil * * * is shown in the following table [by percentages]:

Fine gravel.	Coarse sand.	Medium sand.	Fine sand.	Very fine sand.	Silt.	Clay.
0.1	0.7	0.9	3.3	1.4	41.2	52.3"

The survey of the Woodland Area was made prior to that of the Marysville Area. The mechanical analyses of the two soil types are identical. In reporting on the Woodland Area there could have been no mistake in advising on its adaptability for the crops mentioned, for, at the time the survey was made, the land was producing a very heavy alfalfa crop.

It should be remembered that all information, used in showing the original Marysville Area report of this soil type to be grossly in error, was as available to the Government soil experts as to the writer, and that for their most adverse report on the soil in question there is no real excuse. Nevertheless, as it was obvious that an error had been made, a supplementary report revising the survey of certain soils in the Marysville Area has been issued by the Government, in which the former report was corrected and the mistakes excused on the ground that at the time the survey was made the soil in question was covered

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with water, thereby making thorough investigation difficult! Certainly this was reason enough for failure to get good first-hand data; but why should a report be made condemning many thousands of acres of land on such flimsy investigation?

The writer dissents from Mr. Sellew's comments as to the propriety of including the discharge records at Yuma prior to 1903, in Table 3. All available data should always be given, with explanations, and thus permit the reader to draw his own conclusions according to his own experience and judgment. Furthermore, the result of taking the yearly rainfall for the entire 265 000 sq. miles of water-shed of the Colorado River as the combination of yearly rainfalls at eight different stations scattered over the water-sheds, which it was assumed gave a fair average, does not seem to be justifiable. This is shown by platting the average rainfalls and yearly run-offs, as given by Mr. Sellew for 1903 to 1912, as in Fig. 88. This proves that there is no law that can be deduced connecting yearly rainfall and run-off totals on a water-shed as large as that of the Colorado River, and that one cannot tell with any more certainty than a guess what the run-off of the Colorado River was by using the rainfall data of the eight rain-gauge stations selected.

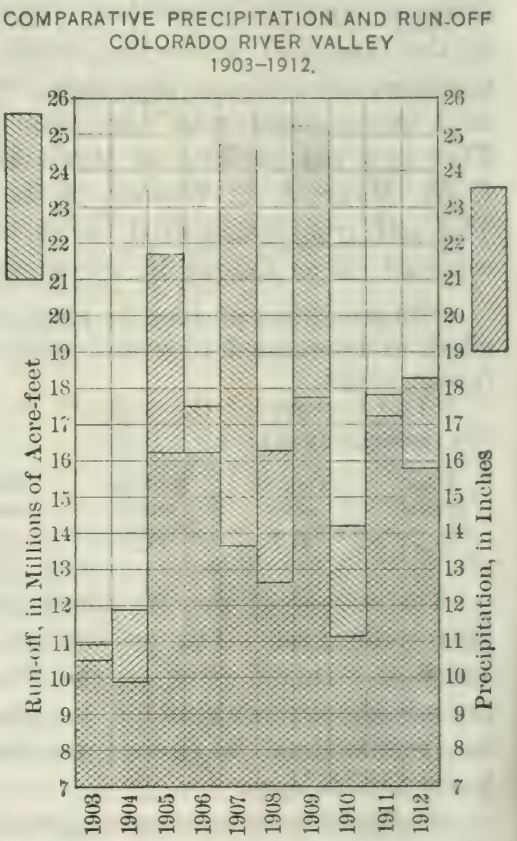


FIG. 88.

Therefore, to condemn the discharge records at Yuma prior to the establishment of permanent gauging stations there, as being too low, because they do not conform closely to what one might expect from the rainfall records given, in the writer's opinion, is wrong. The early data are not precise, but they are valuable and should have weight in the consideration of the problem.

That there may be long cycles of high and low yearly discharge of rivers is well established. The writer calls to mind some investigations which he made into the past levels of the Great Salt Lake. By inquiry from old settlers and by searching records it was found that in the last century Great Salt Lake had two cycles, and probably three, of extreme high and extreme low water, and, according to such

experience, the lake should be high at this time, and such is the case. The lake is now some 8 ft. higher than it was at the time of the construction of the Lucin Cut-off in 1902, and this rise has taken place in spite of the fact that, for irrigation purposes, much more water than formerly is now being diverted from the rivers emptying into the lake. Mr. Marshall

H. T. Cory, M. Am. Soc. C. E. (by letter).—The writer has been surprised and greatly interested at the reception of this paper by those connected with the several interests involved. Though especial effort was made to be fair and dispassionate in the presentation of the various features of the work and situation, yet so much bitterness has been engendered between many of the various agencies in the development of the region that it was felt impossible to write anything worth while and avoid causing, in many quarters, a feeling of unfairness. The discussions by Mr. Sellew and Mr. Chaffey contain all matters of that kind of which the writer has been able to learn, and Mr. Chaffey's discussion deals primarily with matters concerning which the writer at no time had any personal knowledge. The other discussions, numerous conversations, letters from officials of the Southern Pacific Company, of the United States Reclamation Service, of the Interior and State Departments of the National Government, of the California Railroad (Public Utilities) Commission, of the California Development Company and most of the Mutual Water Companies in Imperial Valley, of the Imperial Irrigation District, and from Mr. Rockwood and several of his associates in the project, have all expressed appreciation for the fairness of the presentation. In several cases minor changes in wording have been suggested, but it is a source of much pleasure that all these seemed to be not only fair but an improvement, and every one of these changes has been made. Mr. Cory

The writer must confess that he has often felt very pessimistic and impatient about the unreasonable attitude and stubbornness of a prejudiced man, and therefore such a result is most interesting and encouraging. Possibly, after all, misunderstanding is responsible for much of the failure to appreciate the work and real motives of others. At any rate, this is a striking illustration of men of affairs taking a common ground, or at least a tolerant appreciation of others' actions and opinions, when all the facts of a complex situation are systematically set forth in an apparently fair and comprehensive way, even with the handicap of strong feelings in the matter based on partial or misleading information. If such better understanding will result in less misapplied energy in completing the work thus started, the writer's efforts will be very well repaid.

The next most interesting things brought out by the discussions are the opinions as to the contents of the paper. Mr. Sellew regrets

Mr. Cory.

that the record of engineering operations was "partly obscured by a mass of legal and financial entanglements which properly have no place in a purely engineering article." Mr. Elwood Mead, on the other hand, finds that this same obscuring "mass of legal and financial entanglements" is the most valuable part of the paper, saying:

"The chief value of this paper, however, is not found in its description of the engineering methods used, but in the social and political conditions, attending irrigation development in the West, which it reveals. * * * as these are fundamental features of irrigation development, it is that portion of the paper which the writer proposes to discuss."

Mr. Buck confines his discussion largely to criticizing sharply Mr. Sellew's view. President Swain, in his annual address recently delivered before the Society, urges engineers to feel that their duty is to study and deal with financial, legal, and even social entanglements, and not those only of physics and applied mechanics. The views of various individuals in this regard are no doubt characteristic of their respective holders and the result of their life experiences.

Mr. Robson has added a more detailed study of the evaporation at the Salton Sea and shows that the figures given in the paper were somewhat too large, surprisingly small though they seemed. It might be well to point out that the coefficient, 0.6, derived by Mr. Robson for 6-ft. pans becomes, by Professor Bigelow's formula, 0.49 for 4-ft. pans and 0.4 for 2-ft. pans, in estimating the evaporation from large water surfaces from experimental small-pan data.

Mr. Knowles expresses the feeling, shared by many engineers as well as laymen, that reservoirs on the head-waters and along the tributaries of streams subject to disastrous floods offer the best means of flood control in theory, and oft-times in practice. It is probable, as a matter of fact, that Messrs. Knowles, Sellew, Grunsky, and the writer are not far apart in this matter, and that all agree that the extent to which such methods may be feasible differs with each stream. Mr. Knowles asks for data relative to this question, but it is believed that those presented by Mr. Sellew and the writer are all there are.

Concerning these data, however, it is important to remember that neither Mr. Sellew nor the writer has any personal knowledge of the storage possibilities in the water-shed, and both agree that the available published facts are not sufficient to determine the feasibility of complete, or the extent of practicable, reservoir control of the Colorado River. Consequently, the conclusion reached by Mr. Sellew, as a result of his analysis, that there is but one available reservoir site—Browns Park with a capacity of 2 200 000 acre-ft.—on the entire Colorado River water-shed, is not only startling but quite unwarranted. Furthermore, the statement that "storage on the Gila is out of the question, as this stream carries more silt during its flowing season

than the Colorado," is quite out of accord with the opinions of his superiors in the U. S. Reclamation Service, as the Roosevelt Dam has just been completed, creating a reservoir behind it having a capacity of 1 284 000 acre-ft. at a cost of $3 697 000, according to the first item of Table 23. This reservoir is on the Salt River, one of the main branches of the Gila, and has all the latter's essential characteristics. Mr. Cory.

As a matter of fact, many reservoirs will no doubt be constructed on the Colorado River water-shed, including probably the Kremmling site, in spite of the fact that at present it "is in the hands of interests foreign to irrigation or storage development," and these reservoirs will have considerable beneficial effect in reducing flood flows; it is immaterial whether or not the storage possibilities have been overestimated, as this can only affect the extent of such an influence; but, as Mr. Grunsky says:

"Whatever this may be, it will not change the river problem, except as to its magnitude. * * * The same lack of channel capacity will be felt at high stages on the lower river as is felt to-day, and the same problem of keeping the delta channels of the river on the Gulf slope will confront the river engineers of the future as confronts those of to-day, except only that there will be somewhat less water and correspondingly less silt to be reckoned with."

This is especially true because below Yuma the river is practically a "joint track" for the Colorado and the Gila, which are very different types of streams, but each at times alone causes approximately equal maximum flood heights from Yuma to the Gulf. Of course, the floods on the Gila are flashy and those on the Colorado are long sustained, but that does not signify as much as might be thought, so far as river control is concerned. Fortunately, severe Gila and Colorado floods never occur simultaneously.

As to the quantity of water—the discharge of the Colorado at Yuma—the writer believes that the first half of Table 3 should be given, agreeing with Mr. Marshall that it is always best to set forth all available data, with explanations—as was done in this case—and let the reader decide what to use and what to discard. The recorded discharges prior to November, 1902, are by no means as reliable as those after that date, but that does not mean that they are worthless, though it does mean that they must be used with care and discretion. In this matter the comments of Mr. Grunsky on the reliability of the measurements taken since that time, especially at high stages, are very pertinent.

In any event, it seems clear that, beginning with 1905, the discharge of the river has been markedly greater than normal, and that long periods will occur when the discharge will be markedly less than normal—as is pointed out very well by Mr. Marshall. In the case

Mr. Cory.

he mentions the water is within 4½ ft. of the rail base on miles of the very expensive Lucin Cut-off crossing the Great Salt Lake, and the water is still rising. The relationship between yearly rainfall and run-off, even on very large water-sheds, and especially if few widely scattered points be taken, is by no means constant, as Mr. Marshall says, and as the writer and doubtless many others have learned from time to time to their discomfiture. With the existing data it is not safe to decide in more than a tentative way as to either maximum or minimum quantities. Mr. Sellew's maximum figures as regards river control are probably fairly close to the truth, but, with respect to the minimum for estimating safe irrigable acreage, caution must be used. Fortunately, it will be at least another decade before all the lands easily covered by canals in these regions can be brought under complete cultivation, and the dry cycle of years will probably be well under way, so that the safe limit of land which can be served by the natural flow of the river will be determined in time to avoid excessive development.

With respect to the rise of the river bed at Yuma, Mr. Sellew is correct in the opinion that much more information is necessary before definite relations can be established regarding it. The Southern Pacific Company, however, has kept gauge heights at Yuma since 1878, and these records show a steady rise of the river bed (taking averages), or a steady increase of the river discharge. The reader can judge which is more probably occurring. Also, the old maps on record in the British Museum, in archives at Madrid and Mexico City, and in the Bancroft Library in the University of California, have been carefully studied by Mr. Godfrey Sykes, of the Carnegie Institution, in connection with this very matter, and the final figures he has adopted (which will appear in a volume on the Colorado Desert by the Carnegie Institution, now in press) check well with the "memory of a steamboat captain," Capt. Mellon, of Yuma, quoted by the writer in the paper.

Undoubtedly the difference in elevation at any two points on the river will change as the length of the river between these points is modified by meanderings of the stream; and, also, the two points, Laguna Weir and the conglomerate hills at Yuma, are the only ones on the lower river which may be regarded as definitely fixed—as was pointed out in the paper and again by Mr. Sellew. The latter, however, has mistaken effect for cause when he concludes that the increase of 4 ft. in difference of gauge height of the river at Yuma and Laguna Weir "is plainly caused by the meanders of the river" between these two points. This stretch of river had a normal length of nearly 14 miles, from 1856 to 1909, and this had increased to 17 miles in

1912, because the stream had lowered at Yuma more than 3 ft. as shown by the diagram Fig. 3 (for reasons set forth in the paper), though no change is possible at Laguna Weir. The stream has re-established its normal grade per mile by lengthening itself, and, as the bed at Yuma rises again, a corresponding shortening will follow. Mr. Cory.

It would seem that the comments in the paper as to the design and methods of construction of the Laguna Weir were not clear, either to Mr. Sellew or Mr. Grunsky. Fig. 89 shows diagrammatically a cross-section of the weir as built, and as the writer proposes. The resulting structures would not differ in function, concrete top, crest or foot walls, appearance when completed, or general design in the sense of cross-section above ground.

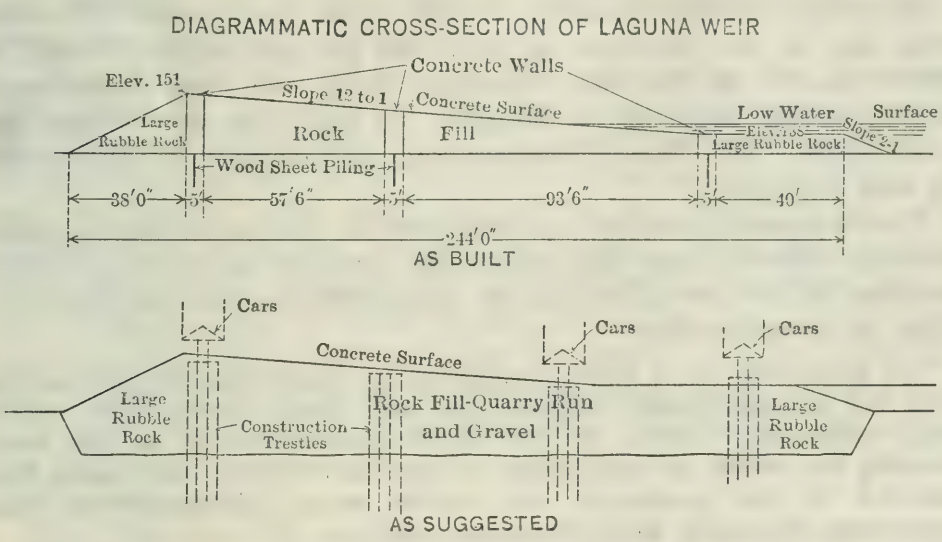


FIG. 89.

The difference is in the omission of the concrete cross-walls (which were very expensive in time and money) in the second design, and changes in method of construction by driving the river sideways ahead of rock and gravel filling from trestles, and thus making it do all the excavation. The result would be a greater yardage of rock in the structure, but the total cost of it in place, including excavation and everything, would certainly not have been any more than the costs given for the material placed in closing the central section of the weir—undoubtedly it would be materially less on account of the much more favorable conditions and the larger quantities. The figures given by E. D. Vincent, M. Am. Soc. C. E., Resident Engineer on the work, and quoted in the paper, are, in round numbers, $1 per yard, made up of the following items*:

* *Engineering News*, June 10th, 1909.

Mr. Cory.

Labor, dumping cars	$0.020
Labor, general	0.020
Sundry supplies and implements	0.008
Railroad hauling	0.217
Depreciation, rolling stock and track	0.062
Excavation, rock-quarrying	0.468
Trestling	0.149
Proportion of storage of apron rock	0.014
Light and water	0.011
Depreciation, special derricks	0.001
Proportion of superintendence	0.032
Overhead charges, Yuma and Washington offices	0.038
	$1.040

Assuming the total quantity of rock in the weir proper to be even twice that in the structure as built, the amount required would be 750 000 cu. yd., or $750 000. Concrete facing and other items would hardly increase this sum to $900,000. As built, the weir proper cost $1 672 168.20. Mr. Clarke's comments on such methods of construction are very pertinent.

This suggested cross-section and method of construction is fundamentally different from that which Mr. Grunsky mentions as recommending in 1896 for a low dam in the Colorado in Iceberg Canyon—a dam to be constructed

"of loose rock on a sand foundation by blanketing the bed of the river for a considerable distance up and down stream with broken rock, using large blocks for the down-stream portions of the work, allowing the water to bury these as deep as it would, and using finer material in the up-stream face, which would ultimately be made impervious or nearly so by the river silt."

Such cross-section design and method of construction is of course entirely independent of Mr. Grunsky's suggestion that the weir top should not have been uniformly high but have had a section, say 800 ft. long, a few feet lower than the rest, to hold the river in a definite position above the weir and to insure sweeping drift of whatsoever size over the structure.

Mr. Clarke's comments on "useless dredging" are most interesting—they differ materially from reports made by those who recommended, built, and are operating the dredge *Imperial* immediately below the Concrete Head-gate. It seems probable that this dredge has not entirely solved the silt problem in the Imperial canal system, as its advocates have enthusiastically proclaimed. Future results will be watched most carefully.

The writer regrets that he did not make clear that no levee failures were caused by the flood of December 7th, 1906, on that portion of the Yuma Project built under Mr. Sellew's direction, for, on taking

charge of the Project, in October, 1906, Mr. Sellew at once ordered muck-ditching under all new levees. This had not been done before, and there were two failures from this cause opposite the Lower Heading on the Project levees, as stated in the paper and illustrated with a photograph, Mr. Sellew to the contrary notwithstanding. The writer remembers this most vividly, because his original design of levees included thorough muck-ditching, which was peremptorily ordered cut out as an avoidable expense, because, up to that time, none had been done in building the levees of the Yuma Project. After the disaster due to the flood of December 7th, 1906, careful examination of the damage to levees on both sides of the river was made and data were collected to show convincingly those not on the "firing line" that, with thorough muck-ditching, all completed levees on the river would have been effective, and the two failures from this cause on the Project levees were considered very significant. Mr. Cory

The writer is inclined to agree with Mr. Sellew that the second break might not have occurred had the borrow-pits been on the river side, because of the peculiar facts of the short duration of that particular flood and the location of the breaks it caused. Nevertheless, the levee would have failed just the same in the several places. Also, muck-ditching would have prevented all trouble. When failure occurs, future construction designed with special reference to preventing that type of failure alone is most unwise, and it was felt that a failure similar to that which happened with the extension of the levees built later by Mr. Ockerson would be quite as disastrous as though caused by lack of muck-ditching or anything else. The design adopted was expected to prove safe from both these dangers, and, thus far, has done so entirely.

It is true that the recommendations of the first and second consulting boards as to levee construction and reconstruction were not followed—a careful reading of these recommendations and of the paper will, it is believed, convince the reader that this was wise. Especial attention is called, for example, to the recommendation on page 1394 to comply with which would have cost $12 000 per mile, though the total cost of the levees, exclusive of gravel blanketing, was considerably less.

As to the efficiency of the interrupted or checkerboard system of borrow-pits, Mr. Grunsky's observations coincide with the writer's, as set forth in the paper. There is no doubt that some—even though slight—additional security is obtained. The question in any case to be decided is whether it is great enough to justify the additional cost. The writer's opinion is that, as the materials grade more and more into those easily eroded, the available precautionary measures are, in their order of effectiveness: river-side borrow-pits with occasional short traverses; river-side continuous borrow-pits; river-side checkerboard

Mr. Cory. borrow-pits; land-side borrow-pits; hydraulic-dredge-built levees, as described by Mr. Grunsky; and, lastly, blanketing with cementing gravel, concrete, etc.

The effectiveness and cost of checkerboard borrow-pits, and consequently whether or not to use them, must be determined for each individual case. With those soils and conditions of vegetation, grade, etc., where they will afford all the needed added security, considerable additional cost is justified; and, where this is not the case, little or no additional expenditures for them is wise. Similarly, as to the cost, with small levees, narrow berms, and easily handled soil—so that the haul is of maximum importance—the additional cost as a percentage will be greater, and *vice versa*. This difference, as set forth in the paper, was actually determined carefully by building adjoining sections of levee with continuous and with checkerboard borrow-pits, using half of one camp on each section and interchanging men and teams daily. Every effort was made to secure otherwise identical conditions, and it is believed with unusual success. The levee was uniformly 8 ft. high, 10 ft. wide on top, slopes, 3 to 1, berms 40 ft., borrow-pits with 50-ft. traverses each 400 ft. in one case, and 100 ft. alternately borrow-pits and undisturbed in the other, depth of pits in both cases, $2\frac{1}{2}$ ft. at levee end and 4 ft. at far end. The material was the Colorado Delta silt described at length in the paper. The costs, excluding contractor's profits, were found to be 19 and $25\frac{3}{4}$ cents, respectively, per yard of embankment in place on completion; the settlement was very slight. This is in the ratio of 100 to 135.5. Mr. Sellew, with essentially similar work, done under normal instead of rush conditions, reports the costs as 11 and 13 cents, respectively—shop costs only—or in the ratio of 100 to 118. If the reader is interested as to the consistency of these two results, he can compute the average haul in the two classes of borrow-pits and check them up.

The method of constructing levees by hydraulic dredges, as suggested by Mr. Grunsky, has been used to a considerable extent on the Sacramento River, although the material from the muck-ditch is there usually deposited on both the river and land sides of the trench, and the hydraulic fill made in the excavation and space between the two banks. The soil in that locality is such that it does not erode easily, and, more often than not, the levees are located on the river banks. With levees set well back and with the shallow, muddy Colorado, it is very doubtful whether such methods could be used successfully for levee building on that river. Furthermore, the amount of work done, especially during any one year, will probably be so small, and the deterioration and cost of handling a dredge on the lower river so great, that the total levee cost would probably be much higher than by the present methods.

Mr. Grunsky points out that the levees along the river have not yet been subjected to a conclusive test, and suggests two points: seepage under the levees with land-side borrow-pits, and reasons why the levees have not yet needed to be as high as were built. As to the former; miniature, pin-head, under-water volcanoes did appear, but their number and size did not seem to be large, when the effective head was greater, and after a time they stopped entirely. In not a single case did any of these become in any way menacing. Furthermore, last year the water-table rose above the ground surface behind the Reservation Levee of the Yuma Project with river-side borrow-pits, covering a considerable area, and seriously interfering with crops. The action is a rise of the underground water, and not seepage under levee structures. Mr. Cory.

The top grade line of the levees built under the writer's direction has proven to be higher than as yet required because of the abnormal overbank flow and cutting off of Nigger Bend in 1907 (as pointed out by Mr. Grunsky), and later by the Abejas Division, which did lower the river at Yuma, as shown by Fig. 3, and at all points below as far as the diversion, Mr. Sellew to the contrary notwithstanding. Indeed, it was this lowering at the Concrete Head-gate, which, with silting up in the Imperial intake, caused the building of the rock weir across the river at that point in 1910, as commented on by Mr. Clarke. Nevertheless, to have built lower levees would have been taking an unjustifiable risk. This is shown most strikingly by what happened with the levees near the Abejas Dam. In this case the levee grade was fixed by reference to the fresh high-water marks of the 1909 flood which had its outlet down the Abejas, but after that closing was to be held in the old river channel. This latter did not scour down as was expected* and, had the levees held, they would have been overtopped in the vicinity of the dam by a 100 000-sec-ft. river that summer. Fig 90† shows the situation there.

Mr. Grunsky believes the control of the Colorado between permanently fixed banks should not—at least for a long time—be projected beyond some agreed point near the Arizona boundary line, and in this he is joined by the Board of Review which reported on the failure of the Ockerson levees, the writer, and most of those who have been on the "firing line." The exceptions are Mr. Ockerson and Mr. Sellew. This is a matter of judgment, and all available data affecting the matter, for which a place could be made in a paper of this kind, have been given.

Mr. Follett confirms the writer's figures of land slope away from the river banks, and suggests the desirability of a secondary levee from

* See statement by Mr. Ockerson, *Engineering News*, Dec. 7th, 1911.

† Taken from House Document 504, 62d Congress, 2d Session.

Mr. Cory.

the sand hills to the C. D. Co. levee, where it is a mile away from the river, with a gate structure where the main canal is crossed. This idea has been given careful consideration; the Board of Review just mentioned has reported that the money such a secondary levee of defense would cost could be more judiciously expended on the principal levees along the river. The writer agrees with the Board of Review in this.

The writer is surprised, on re-reading the paper on receipt of the discussions, to find that so little had been said about bank protection work. This was partly the result of touching as lightly as possible on unsuccessful river control work done by others, partly because of a tacit understanding with other members of the Society as to the contents of the paper and their discussions thereof, and partly because, since holding the levee embankment on the north side of the Second Break in January, 1907, by gravel and rock revetment, as set forth in the paper and in Mr. Clarke's discussion, the proper way to do such work has seemed obvious. Messrs. Grunsky, Hind, Clarke, Herrmann, and the writer went over this matter together in June, 1907, and agreed as to the general procedure on the lower river—practically

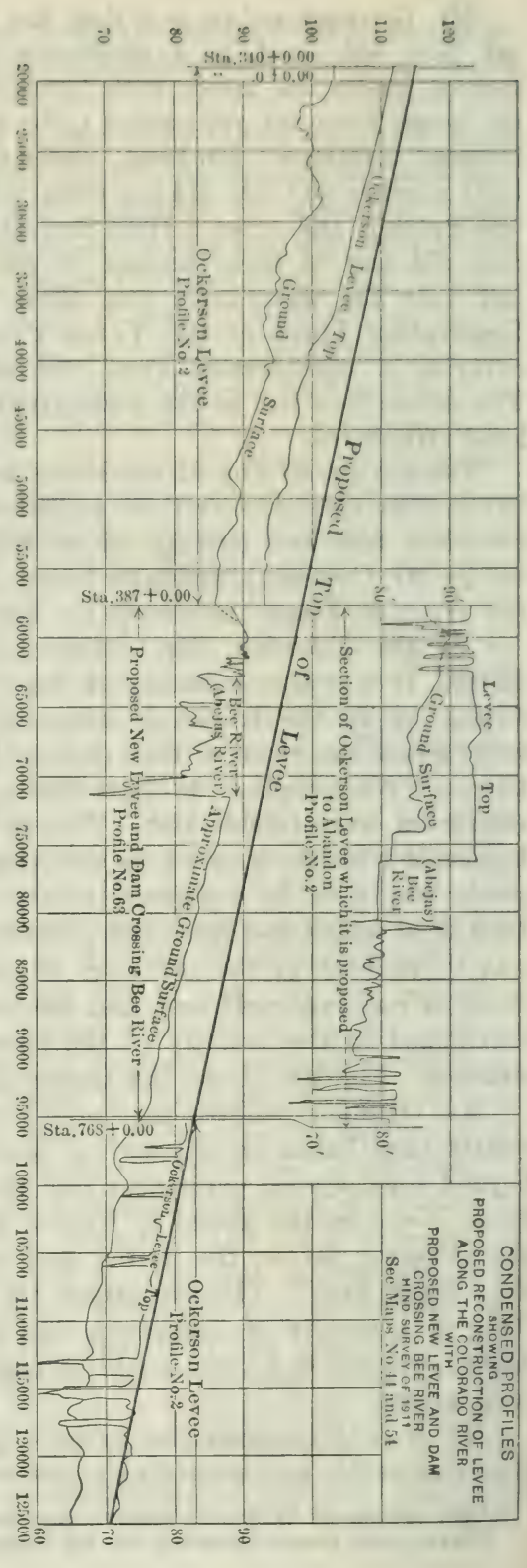


FIG. 90.

that concisely set forth by Mr. Clarke under the heading "Control of the River." Mr. Grunsky reported accordingly to the Secretary of the Interior at the time, and Mr. Sellew has now reached practically the same conclusions, except as to the important difference of reveting only at threatened points and so distributing the work over many years and letting the river do all dredging and placing gravel and rock well down to the limit of scour. In any event, the matter of funds will doubtless distribute the work over a long period, and the procedure recommended helps to arrange the "cutting of the garment according to the cloth." Mr. Cory

With a railroad track on the levees, with large quantities of rock ready in near-by quarries and gravel pits in constant operation, and a very large railway organization which, at any time, on telegraphic notice, will send, for cost plus 10%, work trains, steam shovels, and a completely organized force, the methods of bank and levee production work here seem to the writer to be self-evident. All those who have had to deal with the lower Colorado, except Mr. Ockerson, agree as to the vital importance of a railroad track on these levees.

When thus equipped, and with the foregoing plan of river control and levee revetment, there is no reason for locating levees farther from the bank than needful to provide sufficient waterway for the greatest possible floods. Location farther back means on lower land and hence higher levees and greater difficulty to revet and hold when attacked by bend erosion. Such more distant levee location can at most mean a later date for completing the entire river revetment, and, outside of reasonable limits, this becomes a disadvantage rather than advantage.

Consequently, the writer still feels that:

"Because of the various successful and unsuccessful work done in the region * * *, the Colorado River Delta now presents no unusual, unsolved engineering difficulties; its problems are chiefly matters of statecraft, in both river control and irrigation."

The writer joins with Professor Forbes in regretting that the exceedingly interesting geological sketch and historical review of the Colorado Desert, prepared as a part of this paper, by Professor Blake shortly before his death, could not be utilized in that way. Fortunately, it will appear elsewhere, and its reading is commended to those interested in this subject.

Mr. Chaffey's discussion is of especial interest, not only to the writer and those locally interested, but to many members of the Profession who never had a personal interest in the Lower Colorado. The corrections and criticisms it contains the writer can only admit to be fair; he regrets the inaccuracies in the paper which called them forth, and takes this opportunity to express his pleasure at their being

Mr. Cory.

presented. Since the paper was presented, the Supreme Court of California, in Thayer *vs.* California Development Co., finally held the system of mutual water companies and triparty contracts to be legal, and, in addition, commended such plan. This is the first decision handed down squarely on this point, though in other cases the District Federal Court and the State Supreme Court had by indirection practically decided just the opposite. It is, indeed, a sad state of affairs when it takes from 1906, when the first litigation began, until 1913, for investors and settlers to know definitely whether their operations and organizations are within the law. Mr. Mead is right in saying that at least three-fourths of the irrigation enterprises have been financial disasters because the laws were such that there was no adequate security for the money expended. So it is that, in spite of the legality of the plan as finally decided and its excellent basic business principles, it would doubtless have been better had the project been carried out under the Carey Act.

The writer takes pleasure in expressing appreciation of the standing of Mr. George M. Chaffey in irrigation work in the West. The Ontario Colony he founded in 1883 was selected ten years later as a model for the irrigation exhibit at the World's Exposition, and in his work at Mildura, Australia, he designed, had built in England, and installed, the first centrifugal pumps driven by triple-expansion engines, there being four pumps on the same shaft with a total capacity of 320 cu. ft. per sec. lifted 20 ft. He is at present, among other things, head of the magnificent water system irrigating 10 000 acres of citrus lands near Whittier, Cal., including the highest priced agricultural lands in California ($5 000 per acre). Furthermore, he is a man of affairs and of large means which he acquired principally in irrigation enterprises and banking, and hence has the confidence of moneyed interests.

The comments of Mr. Chaffey and Mr. Marshall on soil examination and reports are pertinent and worthy of serious consideration. The effect of the soil reports on the Imperial region was most serious.

It is nearly a year since the paper was written, and in that time the maze of litigation over the Imperial Irrigation System has, if anything, become more complex. Arrangements have been made to supply water to additional lands by the Receivers of both the American and Mexican Companies for 50 cents per acre-foot, with a 10% free allowance for seepage and evaporation, and with no "water right" or water stock charge or requirement. Thus the later comers are getting water for their lands on better terms than the pioneers in the region, and the opportunity for bettering the financial status of both parent companies by the sale of valuable water rights—and so in the last analysis having the new-comers bear a proper share of the inevitable

burden of readjustment, however that is to be accomplished—has been lost. This resulted from an unfortunate hostility between the two Receivers. Dr. Mead is certainly justified in his comments on laws governing irrigation projects. Mr. Cory.

The development of the country is going on more rapidly than ever, and land values have increased very markedly, due to the growing confidence of the outside public. The 1913 shipment of perishable products alone is 3 880 carloads, as follows:

Cantaloupes	3 499
Watermelons	338
Tomatoes	10
Other vegetables	26
Miscellaneous	7

A new high-line east side main is nearly completed which will water an additional 150 000 acres, and surveys have been completed for a Mexican Canal following closely the Paredones delta divide, crossing New River near Volcano Lake and thence northwesterly to the International Boundary Line near where it is crossed by the West Side Canal. These canals have no especially noteworthy engineering features, however.

No progress whatever seems to have been made in international negotiations looking toward joint governmental river control or conservation and division of the Colorado's waters between the two countries.

A very interesting and complete description of the Colorado River Siphon of the Yuma Project, by George Schobinger, Jun. Am. Soc. C. E.,* has recently been presented before the Society.

*Now Assoc. M. Am. Soc. C. E.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1271

CONSTRUCTION PROBLEMS, DUMBARTON BRIDGE, CENTRAL CALIFORNIA RAILWAY.*

By E. J. Schneider, M. Am. Soc. C. E.

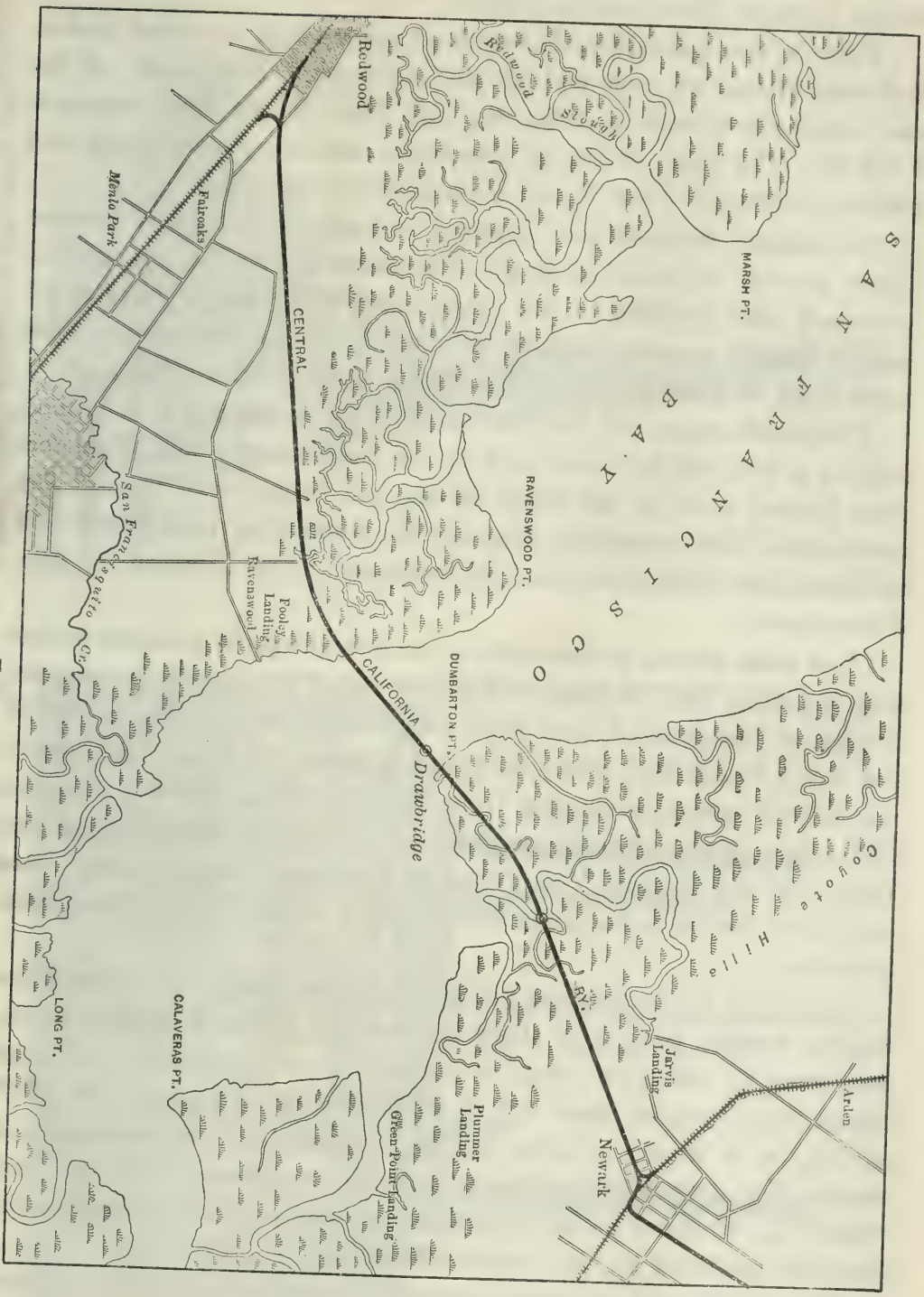
With Discussion by Messrs. L. J. Le Conte, R. D. Coombs, O. E. Hovey, Lewis D. Rights, Theodore Belzner, and E. J. Schneider.

It is not the object of this paper to give a complete description of all the construction features of the Dumbarton Bridge. The aim and purpose is to outline, in a general way, the field methods, and to discuss details only where exceptional problems were met.

To secure an all-rail route into the City of San Francisco from the main line tracks on the eastern shore of San Francisco Bay, thus to avoid ferrying increasing numbers of freight cars between Oakland and San Francisco, the Central California Railway, better known locally as the Dumbarton Cut-off (Fig. 1), was projected and constructed by the Southern Pacific Company during the three years ending September 30th, 1910. The cut-off connects Niles and Newark on the east side with Redwood City on the west side of the bay. Through this improvement, the all-rail distance between Oakland and San Francisco has been reduced 26.1 miles, as compared with the corresponding earlier distance *via* San José.

The bridge proper consists of a draw-span, 310 ft. in length over all, and three 180-ft., double-track, through, riveted, approach spans at each end of the draw. All structural steelwork was designed in accordance with the Harriman Lines Common Standards of 1906,

* Presented at the meeting of March 19th, 1913.


SAN FRANCISCO BAY
Redwood
Redwood Slough
MARSH PT.
Menlo Park
Fairoaks
CENTRAL
RAVENSWOOD PT.
San Francisquito Creek
Ravenswood
Fooley's Landing
CALIFORNIA
DUMBARTON PT.
Drawbridge
Coyote Hill
Jarvis Landing
Arden
Plummer Landing
Newark
Green Point Landing
LONG PT.
CALAVERAS PT.

using what corresponds to a Cooper *E*-55 loading. The center to center distance of end pins is 1 474 ft.

There is also a considerable length of single-track, creosoted timber, ballasted trestle at each end of the steelwork. The length of the east approach trestle is 1 002 ft., and that of the west approach 5 366 ft. The total length of the entire structure, steel spans and timber trestles, is 7 842 ft., or approximately 1½ miles.

No unusual methods were required in building the timber trestles, which are all of standard type. The bents are 15 ft. on centers, and each bent consists of six piles. The piles are of Oregon fir, and in lengths ranging from 60 to 120 ft., the penetrations in some cases being as great as 60 ft., depending on the depth of the mud.

The specifications call for piles of 10-in. tip in the pier foundations, and for a 9-in. tip for those used in the trestles. The butt diameters were limited only by the width of the gin, which was 22 in. The trestle piles were creosoted; those for the foundation piers supporting the structural steel spans were not creosoted. Fig. 2 shows the pile-drivers used.

Test piles, driven at about 300-ft. centers along the proposed bridge axis, indicated that the depth of the mud varied from 2 to 4 ft. near the middle of the channel, and from 16 to 18 ft. toward the shores of the bay. Under the mud was found a stratum of fine black sand, mixed with gravel, averaging from 15 to 20 ft. in depth, and below this a hard clay. It is interesting to note that no trace of clay was found by the Spring Valley Water Company when making tests only about 500 ft. to the north and on a line paralleling the Dumbarton structure.

According to the original plans, it was the intention to extend the wooden trestles from each shore for the full approach lengths to the rest piers of the draw span; but when the east approach had been completed to within about 120 ft. of the draw, on August 21st, 1907, at 3 A. M., a length of about 120 ft. of trestle was undermined and washed out, precipitating the pile-driver equipment into the bay. There was no storm at the time; the accident was due entirely to the currents produced by the outgoing high tide.

These difficulties at once prompted a modification of the original trestle plans. It was first proposed to alter 200 ft. of the east approach and 600 ft. of the west approach, immediately adjoining the

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FIG. 2.—DRIVING PILES FOR DUMBARTON POINT TRESTLE.

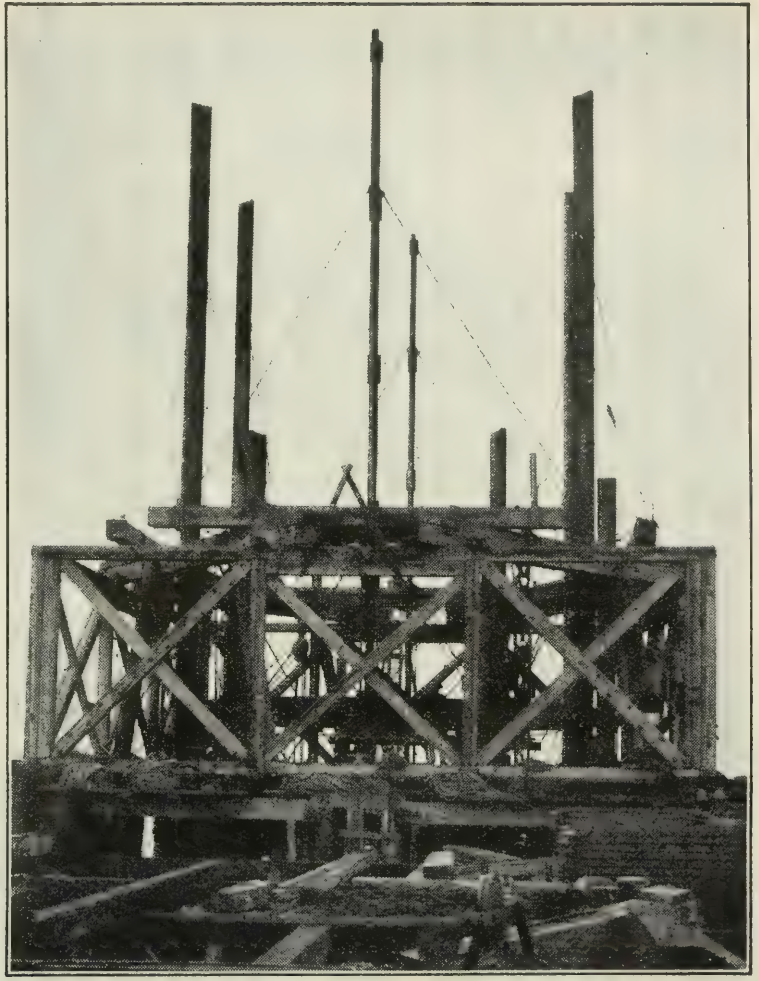

FIG. 3.—ERECTION OF GUIDE-FRAME FOR CYLINDER PIER.

draw span, by supporting the track for these distances on eleven double-track pile bents. After a few of these bents had been driven, it was seen that, by this method, the trestle could not be built steady enough to prevent vibration while high tides were running out. Thus a second modification was decided on, substituting for portions of the trestle approaches, six 180-ft. steel spans, three on each side of the draw (Fig. 8).

At this location in the bay, unusual and difficult foundation problems presented themselves, due to depth of water, the tides, and the nature of the soil. The depth of water at the pivot pier at mean low tide is 51 ft.; at extreme high tide it is 58.3 ft., with an extreme range of tide levels of 11.7 ft. Very rapid currents were caused by the large prisms of water required to pass through the narrow channel, approximately 1 500 ft. wide. Measurements registered a maximum flow of 4½ miles per hour.

These conditions finally led the engineers to adopt cylinder foundations to support the steel superstructure, and these have proved entirely satisfactory. They selected for the type of pier a cylindrical steel shell, enclosing piles and concrete. A description of the 40-ft. pivot pier will illustrate the type, which was also used for all smaller piers. Eight clusters of piles were driven to form a hollow square, 48 ft. on a side, with the center of the square at the center of the pier. About 10 ft. of mud and sand were then dredged, using an orange-peel bucket. The dredged material was placed in a bottom-dropping scow, transported, and dumped into the bay a short distance from the bridge site. Falsework (Figs. 4 and 5), consisting of four wooden trusses, 44 ft. long and 14 ft. high, was constructed of 12 by 12-in. timbers and supported on the pile clusters, for the purpose of sustaining and erecting the guide-frame (Fig. 3) for the steel cylinders. The guide-frame consisted of twelve 20-in., 65-lb. I-beams, 80 ft. long, placed upright to form a regular, twelve-sided polygon about 39 ft. in diameter. These I-beam columns were braced horizontally at intervals of 12 ft. 6 in. by two 12-in. channels placed back to back, as shown by Fig. 6. Additional stiffening was obtained by reinforcing the top and bottom of these 12-in. channels at their mid-points with 8-in. channels. Thus was formed a series of trusses (Fig. 15) which added greatly to the lateral stiffness of the entire frame. In general appearance the guide-

frame resembled the framework for a gas holder (Fig. 9). As the frame was built it was lowered by tackle until it rested on the soil. Then it was weighted to refusal by loading 300 tons of steel on its top. To assist in securing an even settlement, a ⅜-in. water jet was used, while the I-beam columns were tapped with a pile hammer. The guide-frame came to refusal when the first set of horizontal channel struts rested on the sand bottom. These struts were 17 ft. 4 in. above the lowest parts of the frame. The three lower courses (Fig. 7) of the steel cylinder were then assembled on the falsework, and with rope tackles were lowered as a unit into position. A clearance of ½ in. was allowed between the outer flange of the upright I-beams and the webs of the guide-channels which were attached to the shell. A rail was fastened to the top of the section of the shell with clips at each of the twelve points adjacent to the upright I-beam columns of the guide-frame. These rails were used as level rods, being appropriately marked. The shell was driven to refusal, with an even settlement, the bottom edge resting 6 ft. below the lowest horizontal struts of the guide-frame.

The space within the cylinder shell was then further excavated, the center portion being dredged about 5 ft. deeper than at the outside, which just about offset the uplift of the ground, due to the driving of the outside row of piles first and working toward the center with succeeding piles. Two skid pile-drivers, with an overhang of 30 ft., were used (Fig. 11), working from the pier protection used to support them. Inside the steel cylinder 141 piles were driven, at about 32-in. centers, as shown by Fig. 15. Care was taken to place each pile as nearly as possible to match its position on the pile plan. A pile, in position for driving, is shown by Fig. 10. A diver cut off two-thirds of the piles at an average height of 3 ft. above the bottom of the cylinder; the remaining piles extended to within 15 ft. of the top of the pier, or about 13 ft. below the top of the guide-frame.

Before the second unit of the steel cylinder was placed, the lower section was filled with concrete to within 7 ft. of its top, this being the highest level to which the concrete could be placed without disturbance from the tidal currents passing over the top of the section. A bottom-dump bucket was used, having a capacity of 18 cu. ft. Care was taken to provide an even distribution of concrete. A diver was

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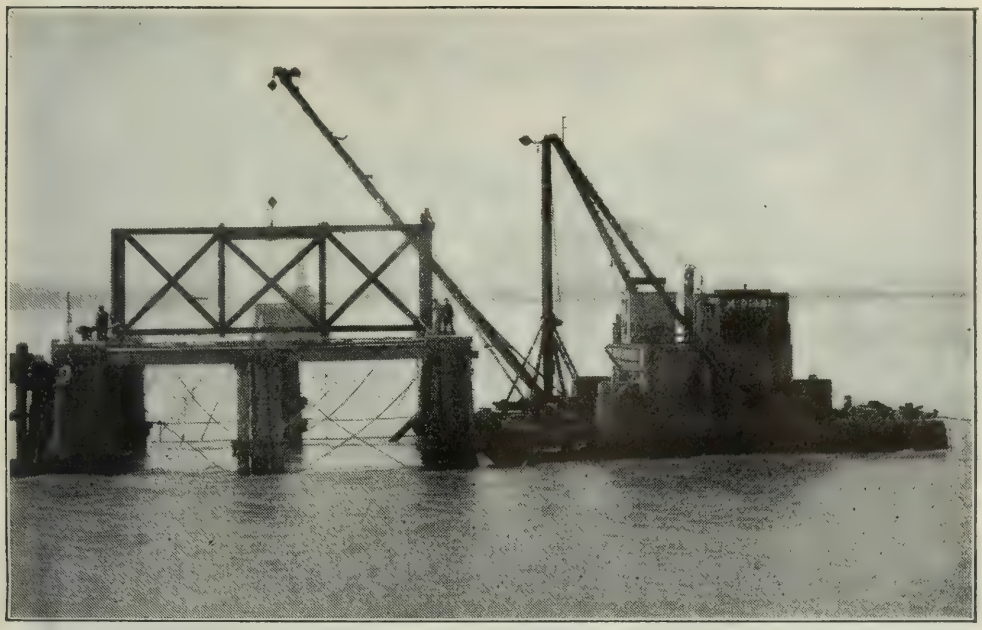

FIG. 4.—ERECTION OF FALSEWORK FOR PIVOT PIER, DUMBARTON POINT DRAW.

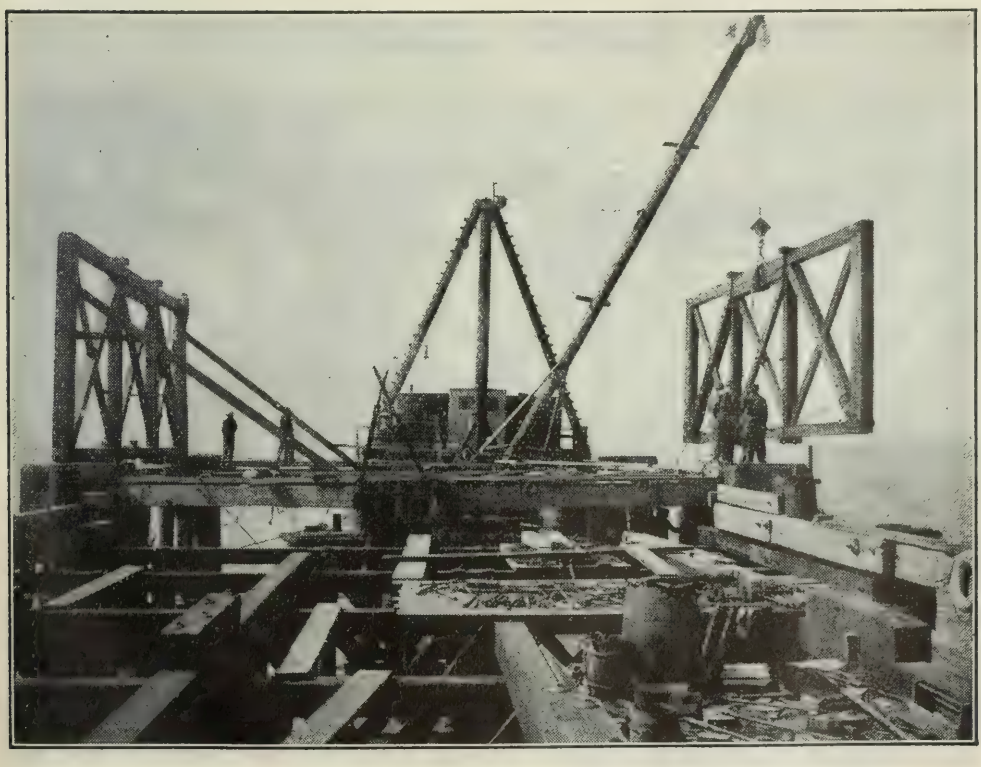

FIG. 5.—ERECTION OF FALSEWORK FOR PIVOT PIER, DUMBARTON POINT DRAW.

15-81

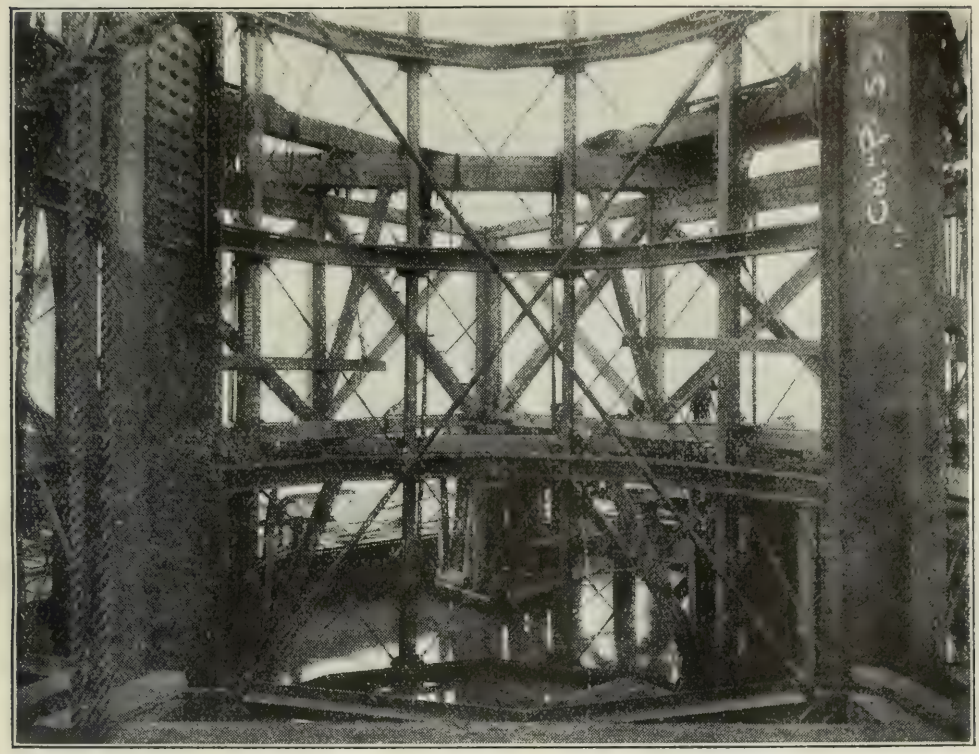

FIG. 6.—INTERIOR OF GUIDE-FRAME FOR PIVOT PIER CASING, DUMBARTON POINT DRAW.

FIG. 7.—THREE LOWER COURSES OF STEEL SHELL FOR CENTER PIER, DUMBARTON POINT DRAW.

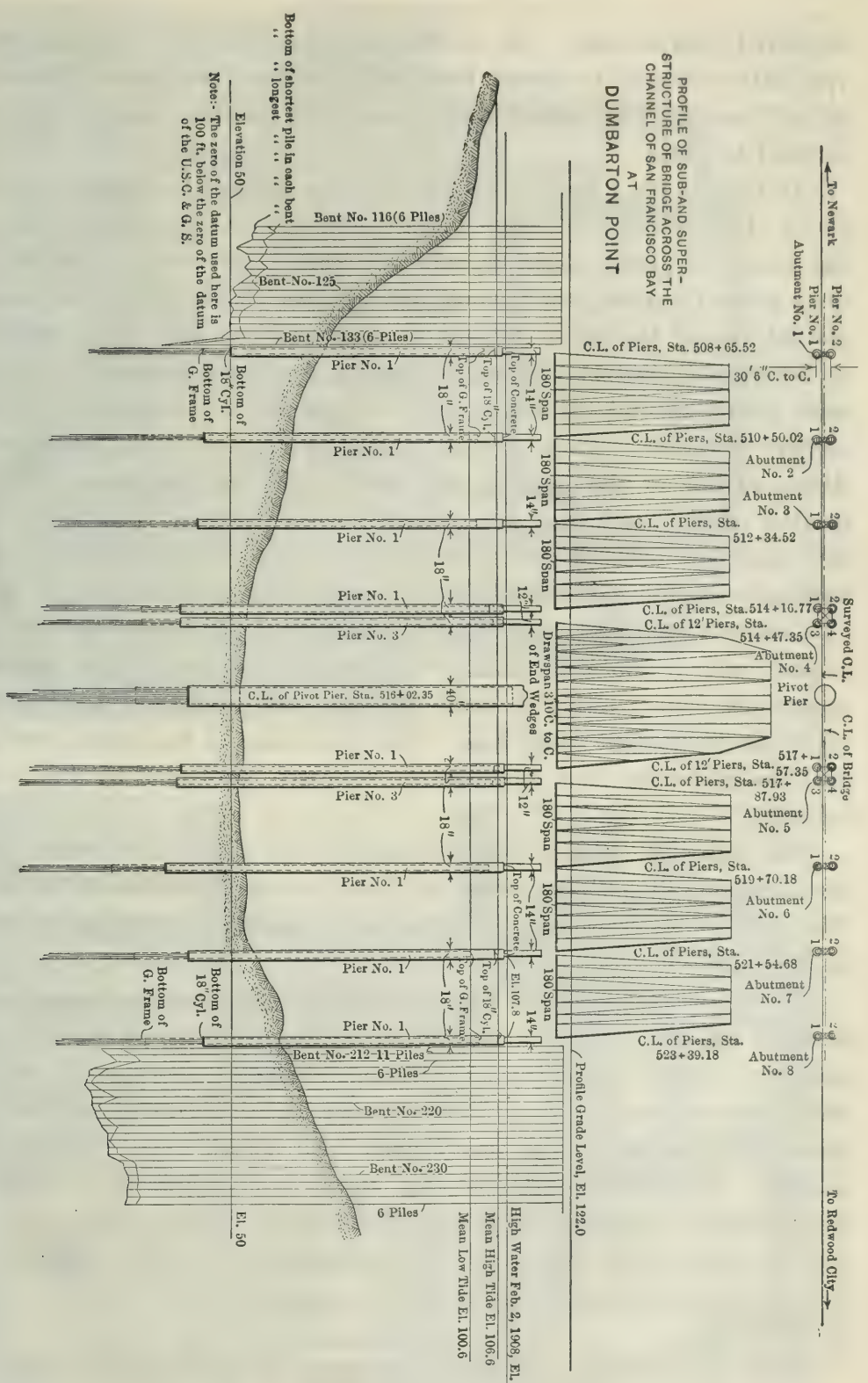
PROFILE OF SUB-AND SUPER-STRUCTURE OF BRIDGE ACROSS THE CHANNEL OF SAN FRANCISCO BAY AT DUMBARTON POINT
To Newark
To Redwood City
Pier No. 2
Pier No. 1
Abutment No. 1
C.L. of Piers, Sta. 508+65.52
30'6" C. to C.
C.L. of Piers, Sta. 510+50.02
Abutment No. 2
Abutment No. 3
C.L. of Piers, Sta. 512+34.52
C.L. of Piers, Sta. 514+16.77
C.L. of 12' Piers, Sta. 514+47.35
Abutment No. 4
Pivot Pier
Surveyed C.L.
C.L. of Bridge
C.L. of 12' Piers, Sta. 517+57.35
C.L. of Piers, Sta. 517+87.93
Abutment No. 5
C.L. of Piers, Sta. 519+70.18
Abutment No. 6
C.L. of Piers, Sta. 521+54.68
Abutment No. 7
C.L. of Piers, Sta. 523+39.18
Abutment No. 8
Profile Grade Level, El. 122.0
Bottom of shortest pile in each bent
" " longest " " " "
Note:- The zero of the datum used here is 100 ft. below the zero of the datum of the U.S.C. & G. S.
Elevation 50
Bent No. 116 (6 Piles)
Bent No. 125
Bent No. 133 (6 Piles)
Bent No. 212-11-Piles
6-Piles
Bent No. 220
Bent No. 230
6 Piles
Pier No. 1
Pier No. 3
Bottom of 18' Cyl.
Bottom of G. Frame
Top of 18' Cyl.
Top of G. Frame
Top of Concrete
180' Span
C.L. of Pivot Pier, Sta. 516+02.35
Drawspan 310' C. to C. of End Wedges
El. 107.8
El. 50
Mean Low Tide El. 100.6
Mean High Tide El. 106.6
High Water Feb. 2, 1908, El. 108.45

FIG. 8.

employed continuously. He found that the concrete materials had very little tendency to spread, because the concrete was being deposited in still water. The second unit of the shell was then assembled and lowered to place.

To provide for a good connection between the units, a rather unique detail (Fig. 16) was evolved. The guide-frame was relied on to bring the joints to proper position and hold them there, but, to insure a more perfect bearing, continuous, flanged-steel plates, ½ in. thick, were riveted around the entire outside of the cylinder on the upper sections at Joints 5, 8, and 11. Flanged clips, of ½-in. steel plates 8 in. long, were placed around the interior of the cylinder, and were 2 ft. from center to center. Thus a butt joint was formed, with cylindrical flange plates on the outside and flange clips on the inside. This portion of the work was also watched very carefully by a diver as the unit was lowered into position, the result being that an even bearing was obtained and also a very satisfactory connection.

Concrete was placed to a level of about 7 ft. from the top of the second unit in the same manner as already described. The various operations were continued until the pier was completed to a level above the permanent water line. After this stage of the work 1 000-lb. coal buckets were used to place the concrete, as shown by Figs. 12 and 13.

The falsework trusses were dismantled and taken down when the last section of the steel cylinder had been lowered into place (Fig. 14). The piles of the falsework dolphins were pulled with the aid of two donkey engines.

The construction for the end abutment piers of the draw span, and also for the piers of the approach spans (Fig. 17), was carried on in approximately the same way as for the pivot pier, so far as work below water was concerned. As no protection was provided for the abutment piers, the foundation piles were driven with a skid driver resting on a scow. This driver (Fig. 18) was elevated on cribwork to provide the necessary clearance above the guide-frames. In the smaller piers only 5 of the 25 piles were cut off at a low level by a diver (Fig. 19); the remaining 20 piles were allowed to extend well up into the top of the shell.

With the exception of the pivot pier, all foundation cylinders were 18 ft. in diameter to an elevation 2 ft. above high water. Above

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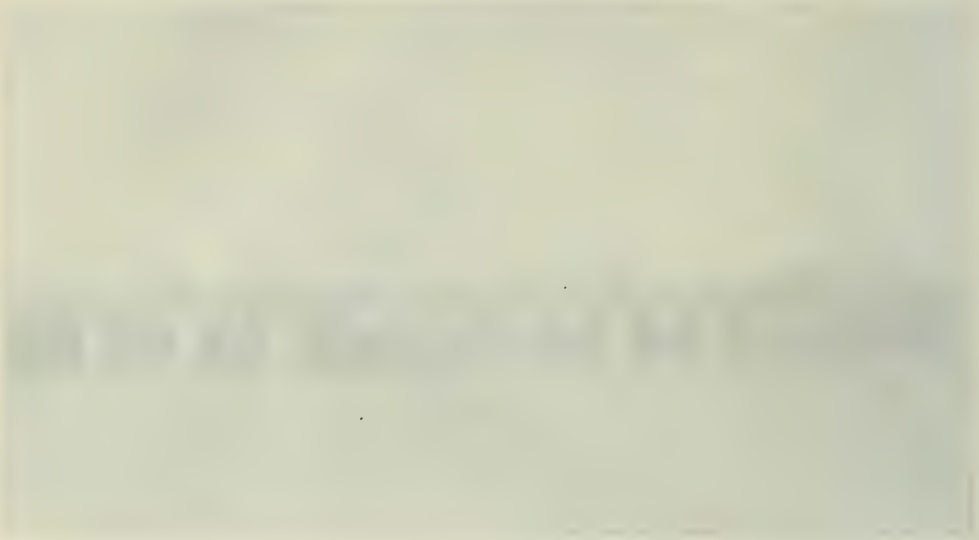
FIG. 11.—DRIVING FOUNDATION PILES FOR PIVOT PIER, DUMBARTON POINT DRAW.

FIG. 12.—PLACING CONCRETE IN PIVOT PIER, DUMBARTON POINT DRAW.



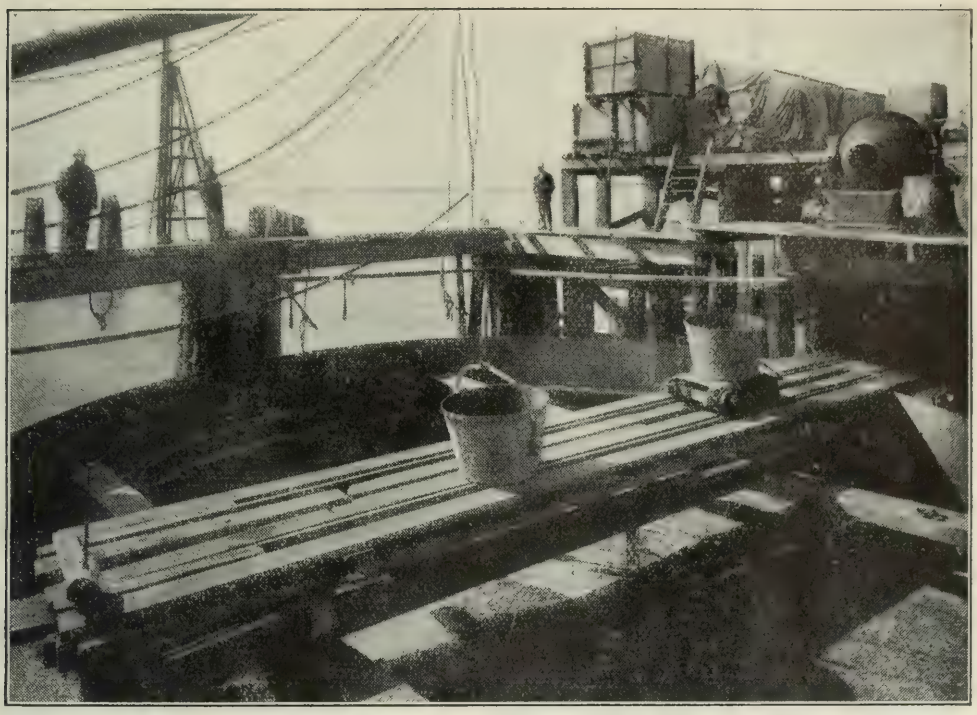


FIG. 13.—CONCRETE PLANT, PIVOT PIER, DUMBARTON POINT DRAW.

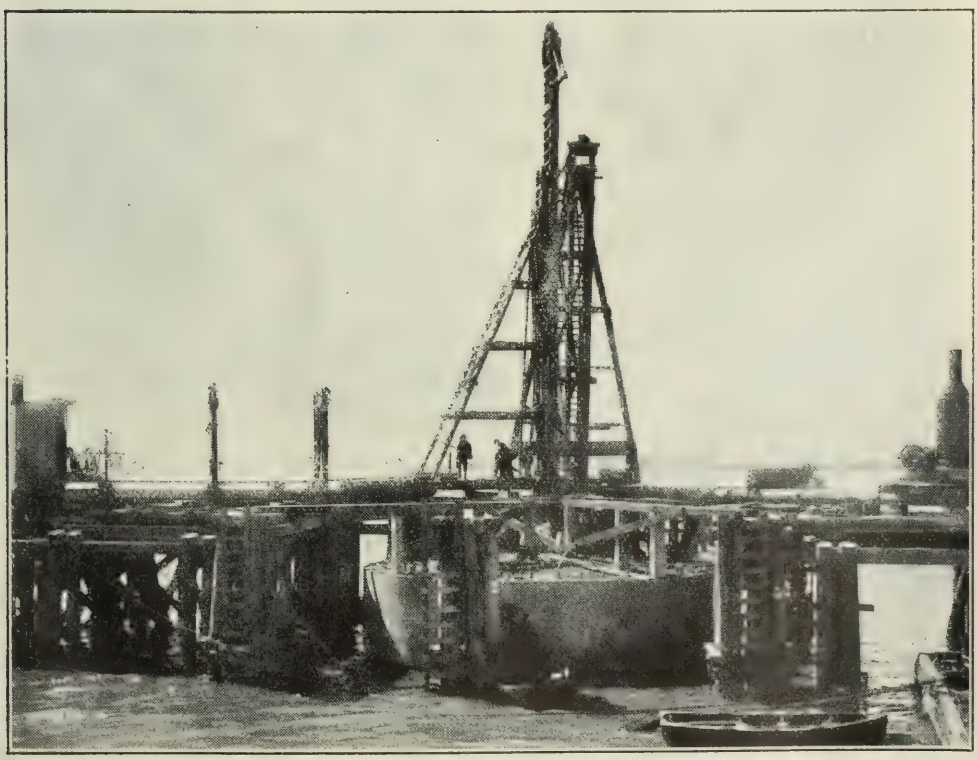
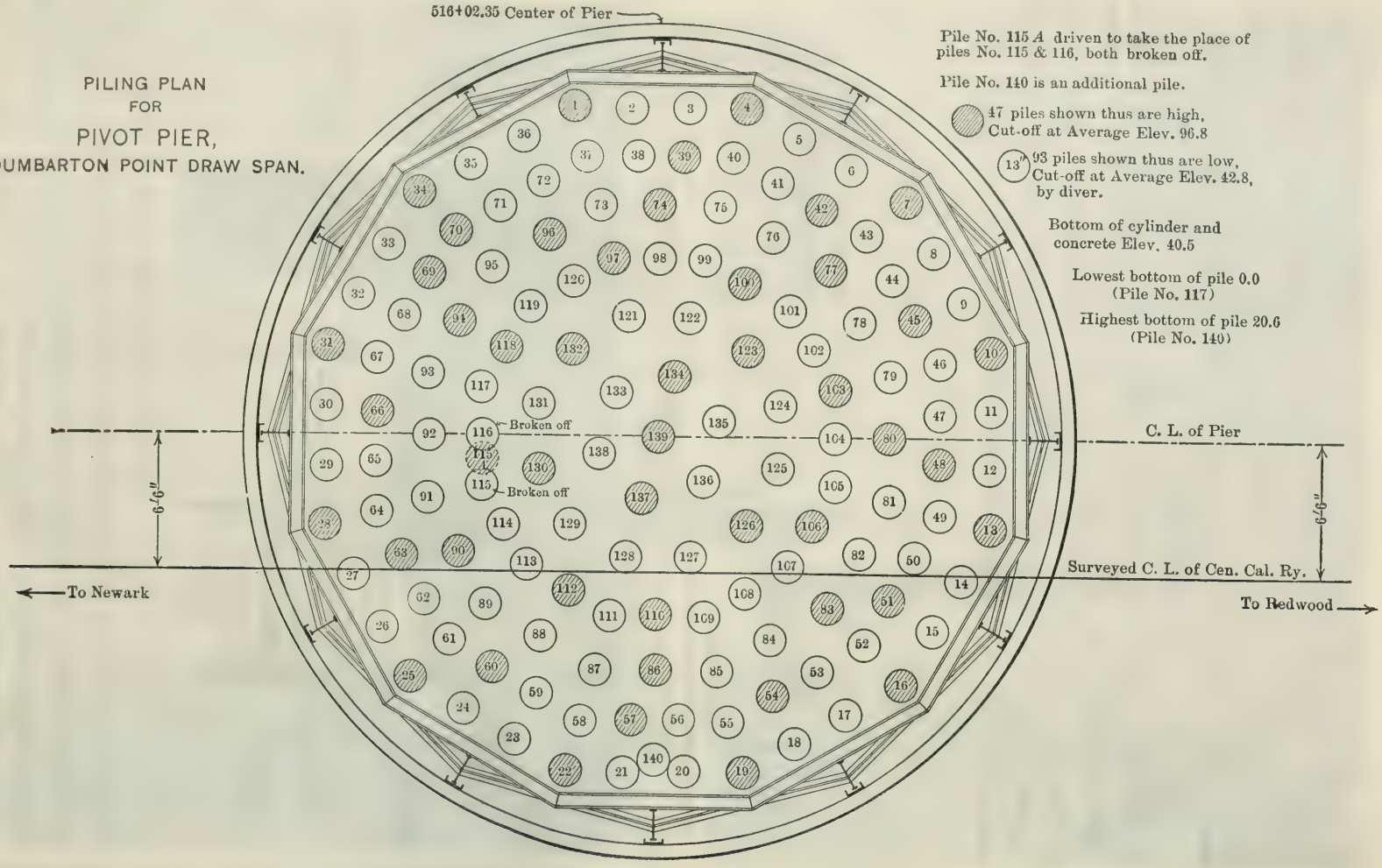


FIG. 14.—PULLING FALSEWORK DOLPHIN PILES, DUMBARTON POINT DRAW.

PILING PLAN
FOR
PIVOT PIER,
DUMBARTON POINT DRAW SPAN.


516+02.35 Center of Pier
Pile No. 115 A driven to take the place of piles No. 115 & 116, both broken off.
Pile No. 140 is an additional pile.
47 piles shown thus are high, Cut-off at Average Elev. 96.8
93 piles shown thus are low, Cut-off at Average Elev. 42.8, by diver.
Bottom of cylinder and concrete Elev. 40.5
Lowest bottom of pile 0.0 (Pile No. 117)
Highest bottom of pile 20.6 (Pile No. 140)
Broken off
Broken off
C. L. of Pier
Surveyed C. L. of Cen. Cal. Ry.
To Newark
To Redwood
6'-6"
6'-6"

this level the diameter for the steel shells of the abutment piers for the draw span was reduced to 12 ft. (Fig. 20); for the cylinders under the fixed spans the diameter was reduced to 14 ft. These reductions in diameter permitted of an easy adjustment due to errors in alignment in placing the lower shells (Fig. 23).

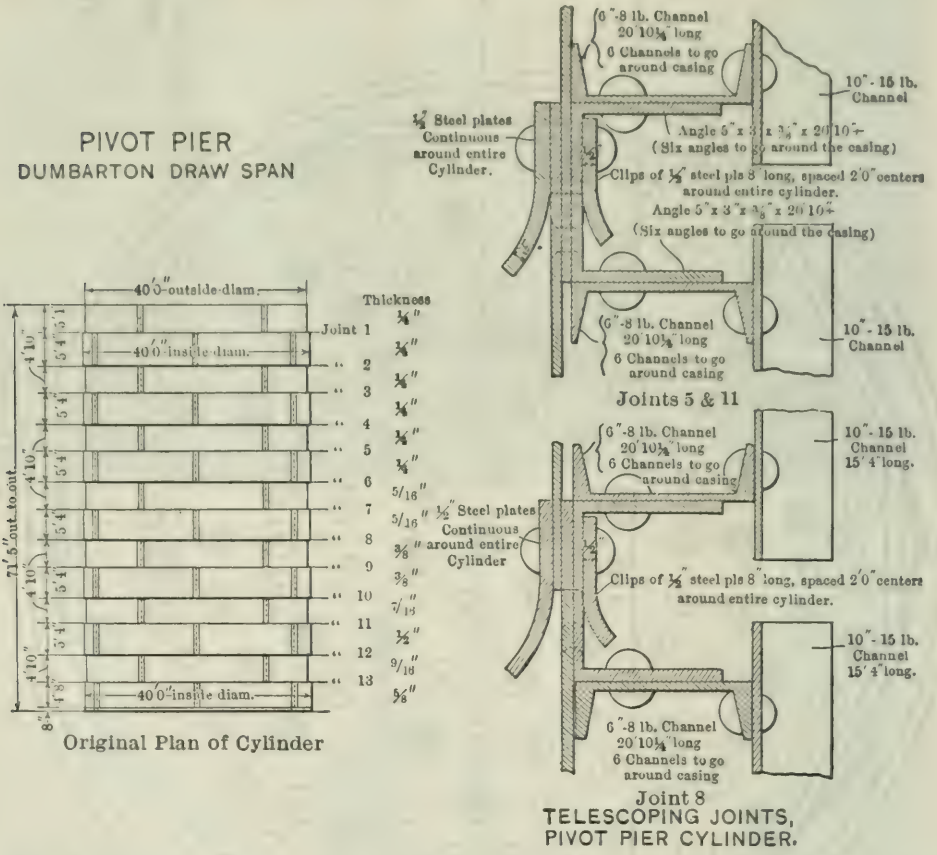

PIVOT PIER
DUMBARTON DRAW SPAN
40′0″ outside diam.
40′0″ inside diam.
71′5″ out to out
Thickness
Joint 1 ¼″
" 2 ¼″
" 3 ¼″
" 4 ¼″
" 5 ¼″
" 6 5/16″
" 7 5/16″
" 8 ⅜″
" 9 ⅜″
" 10 7/16″
" 11 ½″
" 12 9/16″
" 13 ⅝″
40′0″ inside diam.
Original Plan of Cylinder
6″-8 lb. Channel 20′10¼″ long 6 Channels to go around casing
10″-15 lb. Channel
½″ Steel plates Continuous around entire Cylinder.
Angle 5″ x 3″ x ⅜″ x 20′10″ (Six angles to go around the casing)
Clips of ½″ steel pls 8″ long, spaced 2′0″ centers around entire cylinder.
Joints 5 & 11
10″-15 lb. Channel 15′4″ long.
½″ Steel plates Continuous around entire Cylinder
Joint 8
TELESCOPING JOINTS,
PIVOT PIER CYLINDER.

FIG. 16.

To insure a good bond in the cylinder piers at the sections where the 18-ft. diameter was reduced to 12 or 14 ft., fifteen 10-ft. lengths of old steel rail were placed vertically in each cylinder, and spaced around a circumference about 15 in. inside of the upper cylinder. These rails were lashed to the pier girders by steel cables, holes being cut through the webs of the girders where required.

Each of the two rest piers of the draw span consisted of four cylinders (Fig. 21). Above the water line the 12-ft. sections were braced and tied together by four plate girders, each 96 in. in depth. The two girders running parallel to the track line extended through and were riveted to the farther sides of the cylinders. The

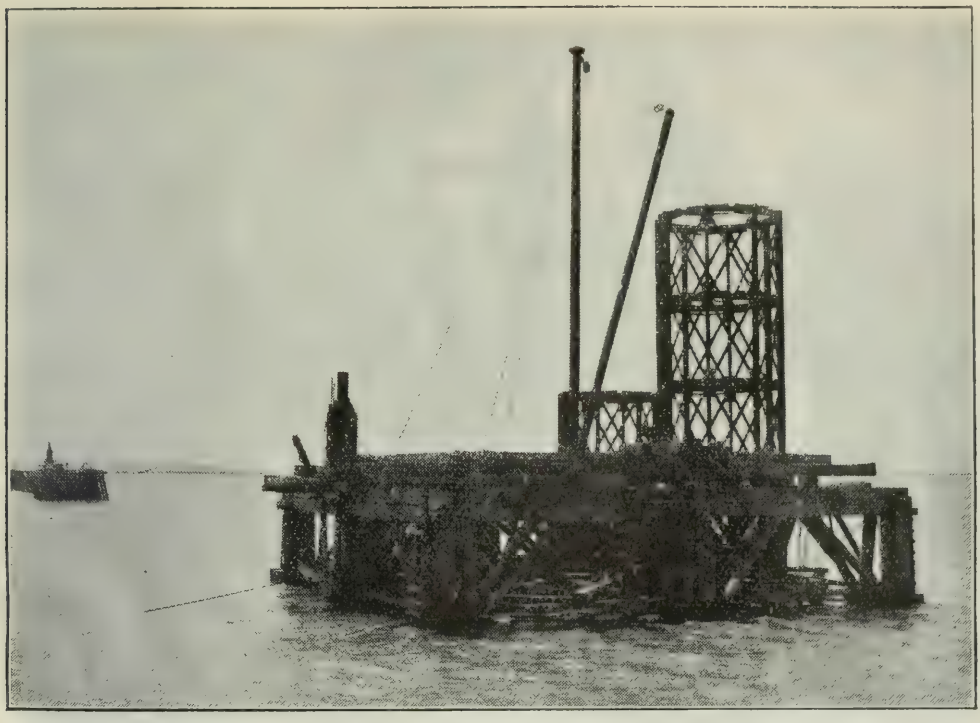


FIG. 17.—ERECTION OF GUIDE-FRAMES, WEST ABUTMENT PIER, DUMBARTON POINT DRAW.

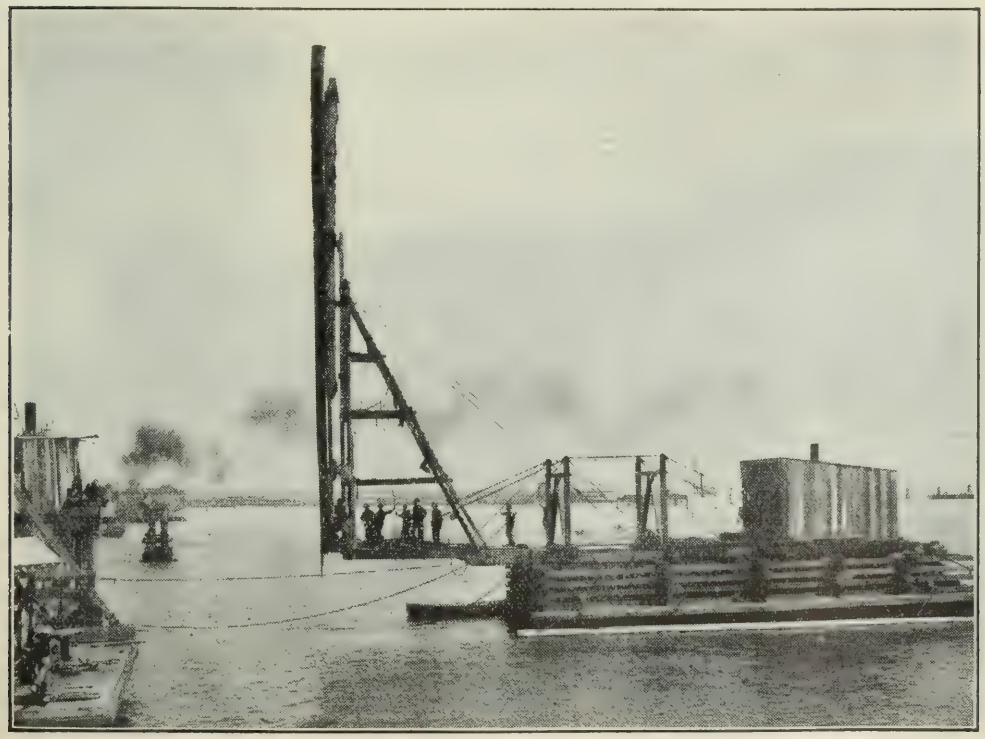


FIG. 18.—SKID PILE-DRIVER ON SCOW, WITH PILE CUT-OFF IN GINS, EAST ABUTMENT, DUMBARTON POINT DRAW.

15-95

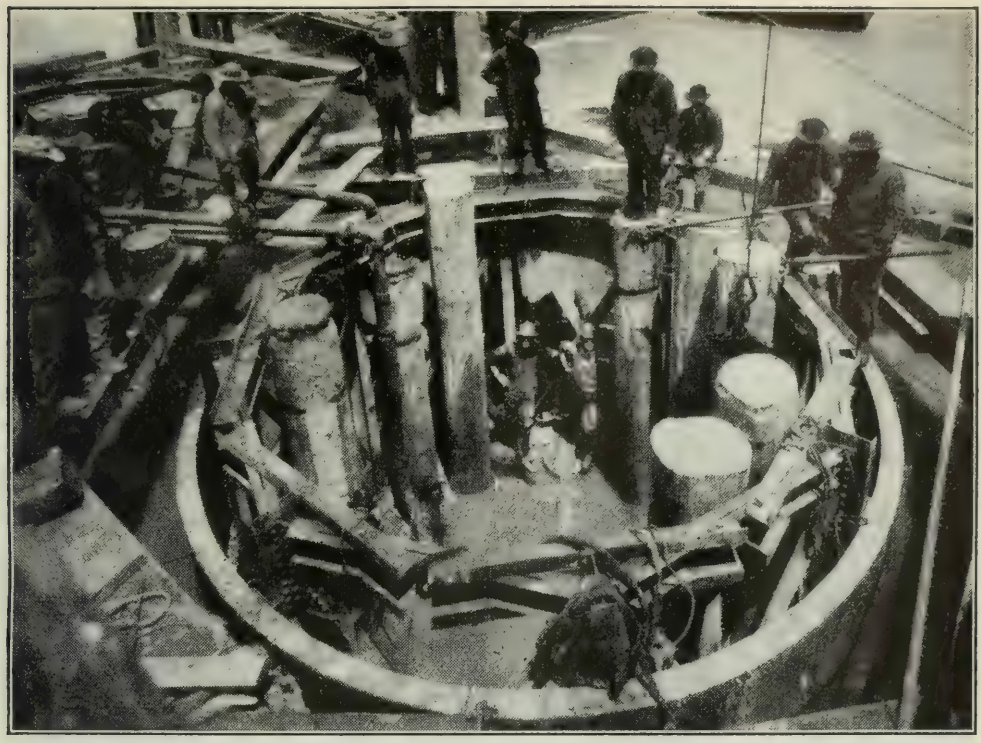


FIG. 19.—CUTTING OFF PILES IN PIER 2, ABUTMENT 4, DUMBARTON POINT.

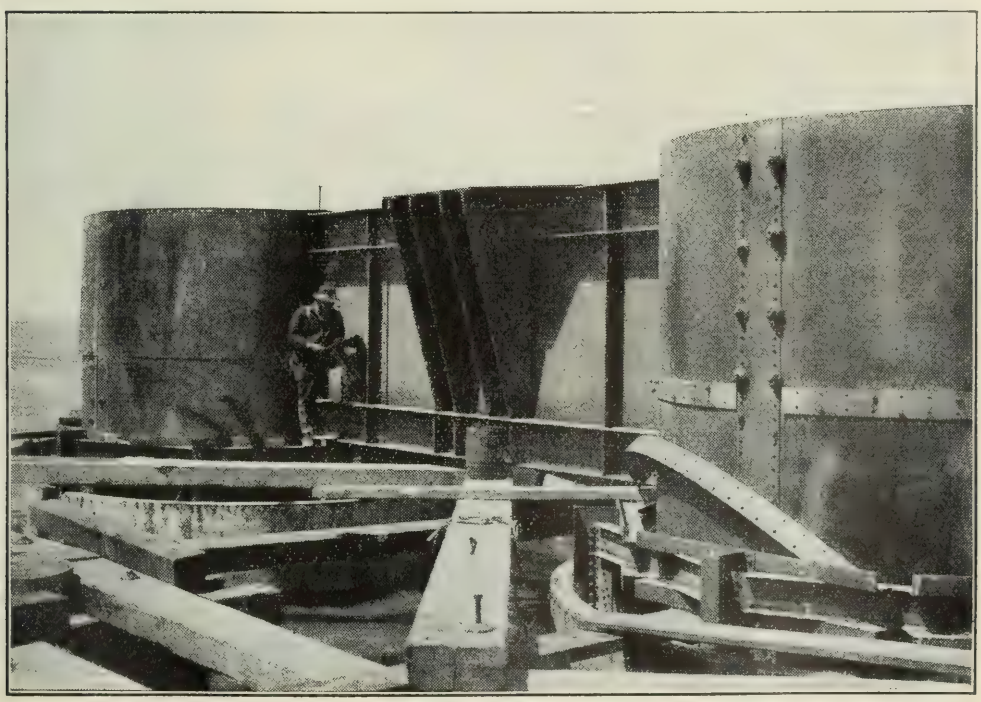


FIG. 20.—CYLINDERS IN PLACE, ABUTMENT 4, DUMBARTON POINT.

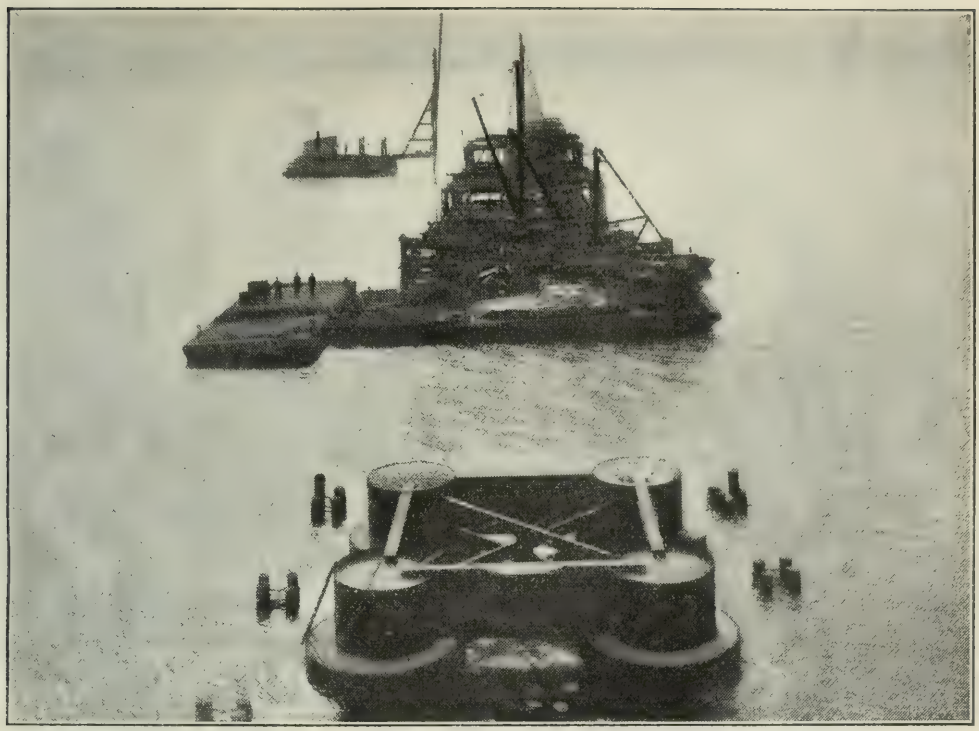


FIG. 21.—LOOKING WEST FROM ENGINE-HOUSE, ON DUMBARTON POINT DRAW-SPAN.

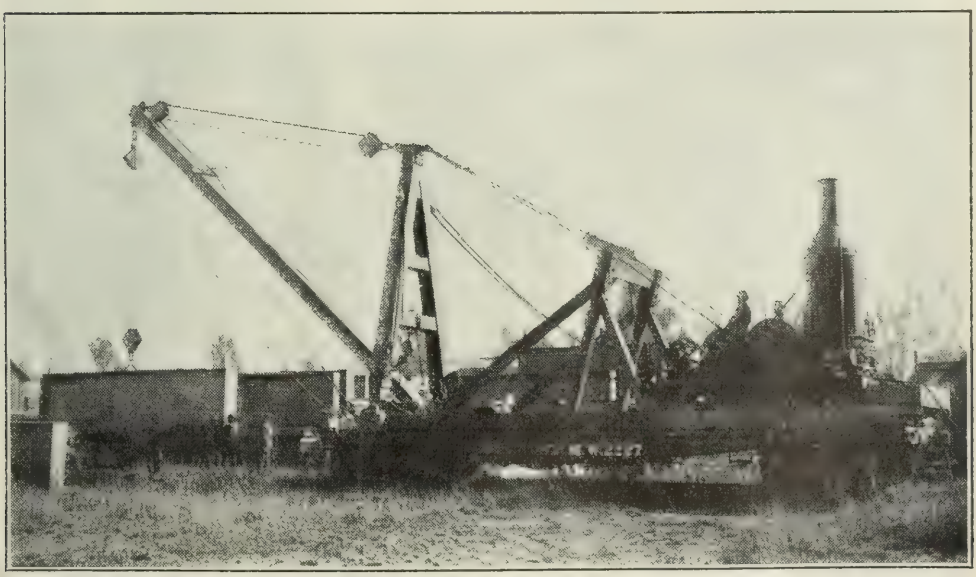
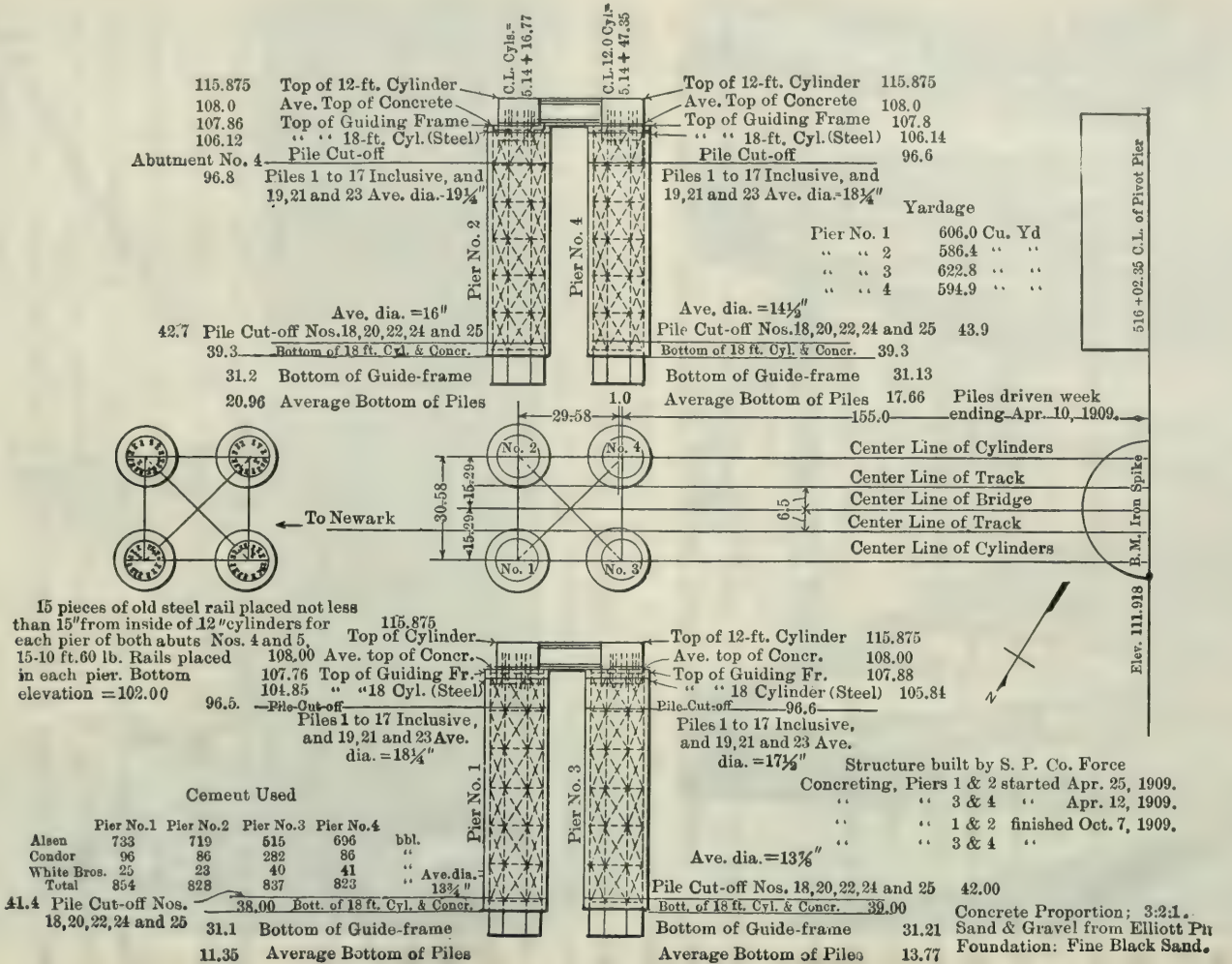


FIG. 22.—DERRICK CAR, HANDLING BRIDGE MATERIAL.


115.875 Top of 12-ft. Cylinder
108.0 Ave. Top of Concrete
107.86 Top of Guiding Frame
106.12 " " 18-ft. Cyl. (Steel)
Abutment No. 4
Pile Cut-off
96.8 Piles 1 to 17 Inclusive, and 19,21 and 23 Ave. dia.-19½"
C.L. Cyls.= 5.14 + 16.77
C.L. 12.0 Cyl.= 5.14 + 47.35
Pier No. 2
Pier No. 4
Top of 12-ft. Cylinder 115.875
Ave. Top of Concrete 108.0
Top of Guiding Frame 107.8
" " 18-ft. Cyl. (Steel) 106.14
Pile Cut-off 96.6
Piles 1 to 17 Inclusive, and 19,21 and 23 Ave. dia.-18¼"
Yardage
Pier No. 1 606.0 Cu. Yd
" " 2 586.4 " "
" " 3 622.8 " "
" " 4 594.9 " "
516 + 02.35 C.L. of Pivot Pier
Ave. dia. =16"
42.7 Pile Cut-off Nos. 18,20,22,24 and 25
39.3 Bottom of 18 ft. Cyl. & Concr.
31.2 Bottom of Guide-frame
20.96 Average Bottom of Piles
Ave. dia. =14½"
Pile Cut-off Nos. 18,20,22,24 and 25 43.9
Bottom of 18 ft. Cyl. & Concr. 39.3
Bottom of Guide-frame 31.13
1.0
Average Bottom of Piles 17.66
155.0
Piles driven week ending Apr. 10, 1909.
29.58
30.58
15.29
15.29
No. 2
No. 4
No. 1
No. 3
6.5
To Newark
Center Line of Cylinders
Center Line of Track
Center Line of Bridge
Center Line of Track
Center Line of Cylinders
B.M. Iron Spike
Elev. 111.918
15 pieces of old steel rail placed not less than 15" from inside of 12" cylinders for each pier of both abuts Nos. 4 and 5, 15-10 ft. 60 lb. Rails placed in each pier. Bottom elevation =102.00
115.875 Top of Cylinder
108.00 Ave. top of Concr.
107.76 Top of Guiding Fr.
104.85 " "18 Cyl. (Steel)
96.5. Pile Cut-off
Piles 1 to 17 Inclusive, and 19,21 and 23 Ave. dia. =18¼"
Top of 12-ft. Cylinder 115.875
Ave. top of Concr. 108.00
Top of Guiding Fr. 107.88
" " 18 Cylinder (Steel) 105.84
Pile Cut-off 96.6
Piles 1 to 17 Inclusive, and 19,21 and 23 Ave. dia. =17⅞"
Structure built by S. P. Co. Force
Concreting, Piers 1 & 2 started Apr. 25, 1909.
" " 3 & 4 " Apr. 12, 1909.
" " 1 & 2 finished Oct. 7, 1909.
" " 3 & 4 "
Cement Used
Pier No.1 Pier No.2 Pier No.3 Pier No.4
Alsen 733 719 515 696 bbl.
Condor 96 86 282 86 "
White Bros. 25 23 40 41 "
Total 854 828 837 823 "
Ave. dia.-13¾"
41.4 Pile Cut-off Nos. 18,20,22,24 and 25
38.00 Bott. of 18 ft. Cyl. & Concr.
31.1 Bottom of Guide-frame
11.35 Average Bottom of Piles
Pier No. 1
Pier No. 3
Pile Cut-off Nos. 18,20,22,24 and 25 42.00
Bott. of 18 ft. Cyl. & Concr. 39.00
Bottom of Guide-frame 31.21
Average Bottom of Piles 13.77
Concrete Proportion; 3:2:1.
Sand & Gravel from Elliott Pit
Foundation: Fine Black Sand.

two cross-girders were framed into the longitudinal ones. To provide clearance for placing the grillage beams under the bridge shoes, it was necessary to cut from each end of each girder a portion of the top flange and web.

Each pier under the approach spans consisted of two cylinders, the lower sections 18 ft. in diameter, the upper ones 14 ft. in diameter, as already explained. The 14-ft. sections were tied together by two plate girders, each 7 ft. 2 in. in depth.

The anchor-bolts for all spans were placed as the concrete was poured. The only provision made for adjustment was that provided by the slotted holes of the shoes.

The erection of the draw span required no unusual methods. The material as it arrived from the plant was unloaded along the tracks, so that it could be placed on a barge as required. The unloading derrick (Fig. 22) was somewhat out of the ordinary in that the A-frame was pin-connected, which permitted it to be raised or lowered, an important and advantageous feature in railroad work. The draw-span trusses, floor, and bracing, were assembled and erected on the draw-span protection in a position at right angles to the line of track. A double derrick or traveler (Fig. 24), running on four lines of rails, enabled the erecting crew to make rapid progress. The span was assembled complete (Fig. 25), ready for riveting, in 14 days. Figs. 26 and 27 show the draw-span.

The six approach spans were floated into position. To facilitate their erection on land, pile bents were driven in the shallow water of the bay shore on the Newark side. A sufficient number of pile bents of this character were built to permit the assembling of two spans at once (Fig. 28). When one span was thus assembled and completely riveted, the temporary trusses of the falsework were knocked out and barges were placed in position. The old steamer *Thoroughfare,* which for many years had been used to ferry freight cars across San Francisco Bay, incidentally having been condemned by the Government, was cut in two, bulkheaded, and, as though by the irony of fate, assisted in the erection of the bridge which is destined to relegate to the past the scheme of bay transportation, for which the old steamer was originally built.

Particular care was used in selecting the time to float the various approach spans into position on the cylinder piers. At the erection

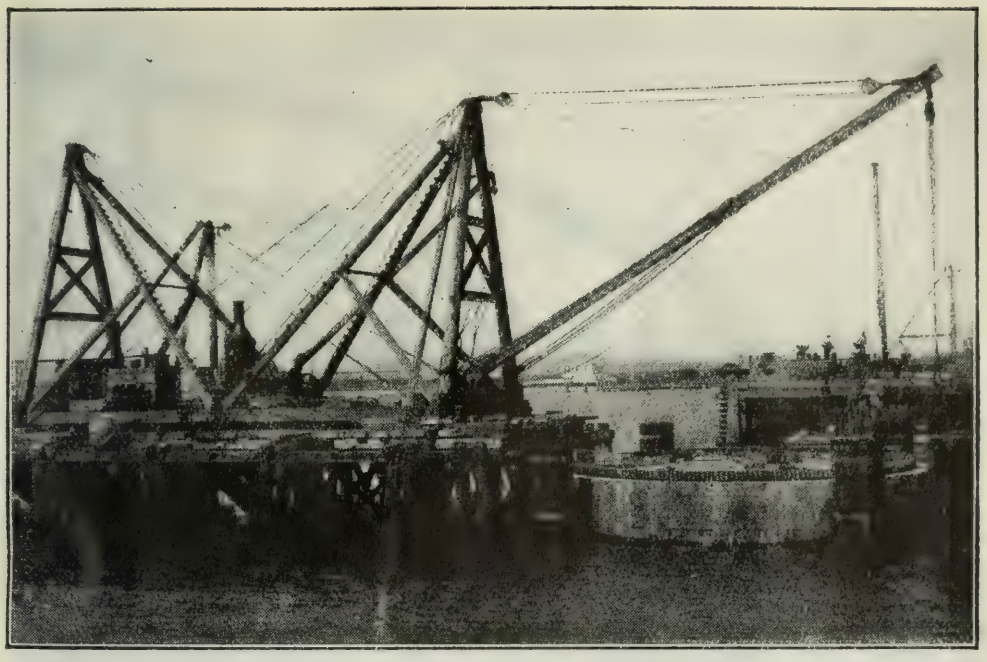


FIG. 24.—DOUBLE DERRICK TRAVELER FOR ERECTION OF DUMBARTON POINT DRAW.

FIG. 25.—ERECTION OF DUMBARTON POINT DRAW.

FIG. 26.—ERECTION OF DUMBARTON POINT DRAW.

FIG. 27.—DUMBARTON POINT DRAW.

1603

FIG. 28.—FLOATING SPAN 5 INTO POSITION, DUMBARTON POINT BRIDGE.

FIG. 29.—FLOATING SPAN 1 INTO POSITION, DUMBARTON POINT BRIDGE.

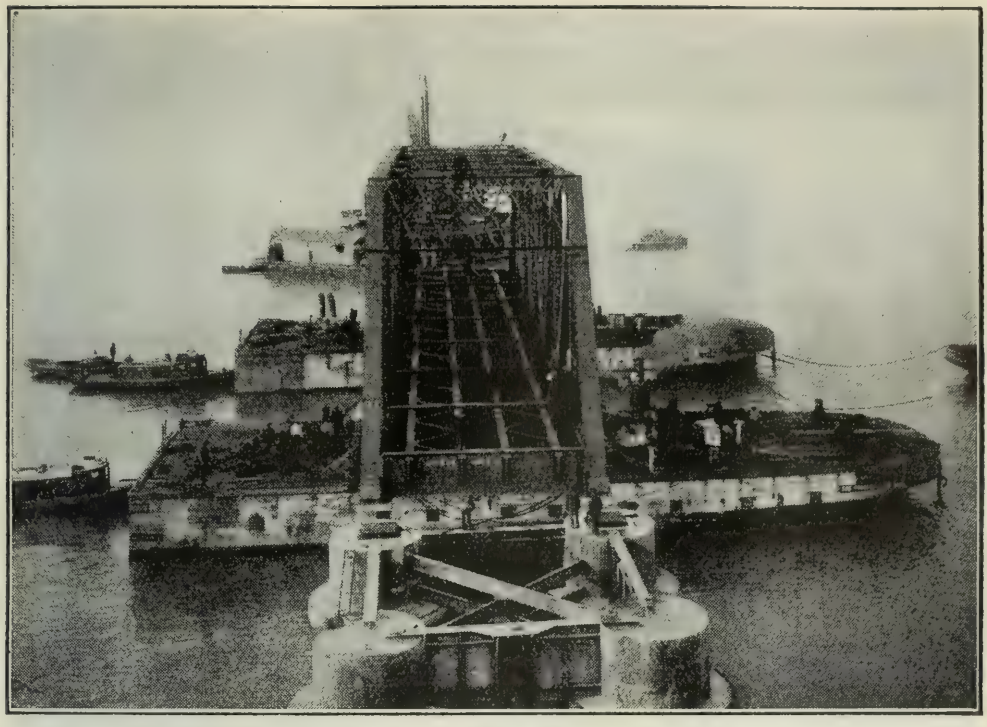


FIG. 30.—FLOATING SPAN 5 INTO POSITION, DUMBARTON POINT BRIDGE.

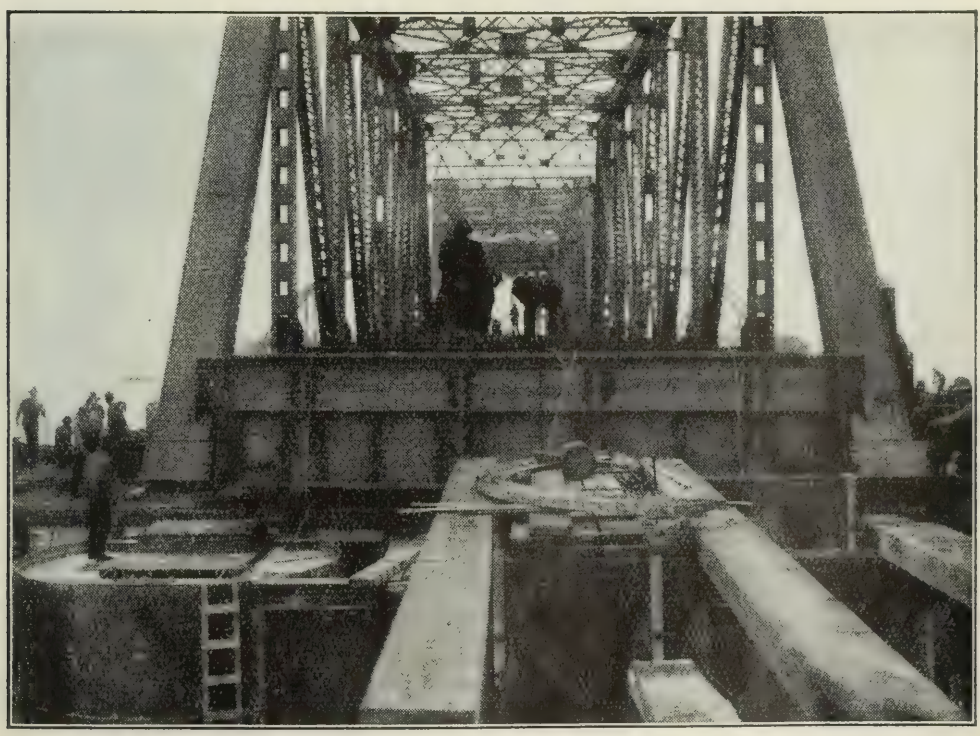


FIG. 31.—FIXED END OF SPAN 1, DUMBARTON POINT BRIDGE.

1604

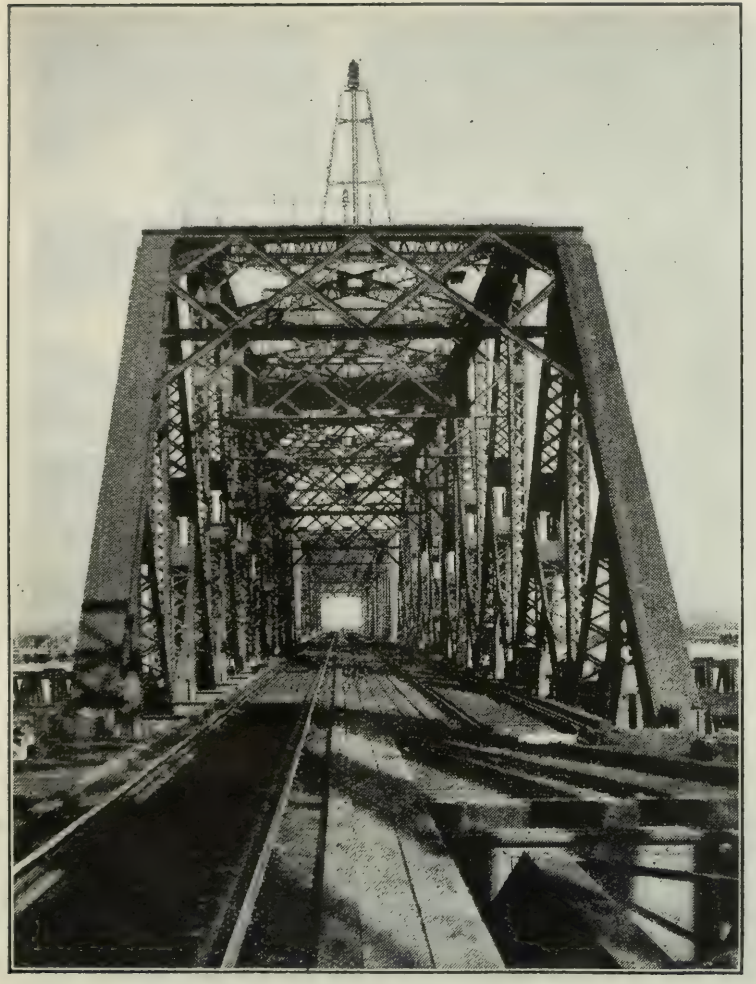


FIG. 32.—DUMBARTON POINT DRAW-SPAN.

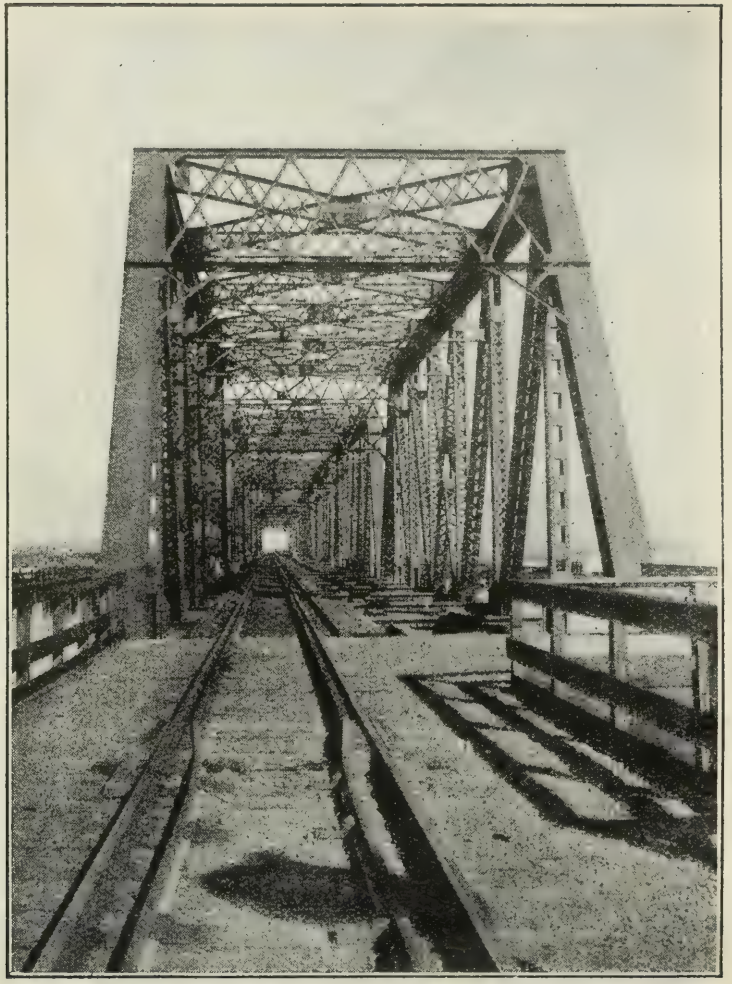


FIG. 33.—DUMBARTON POINT BRIDGE.

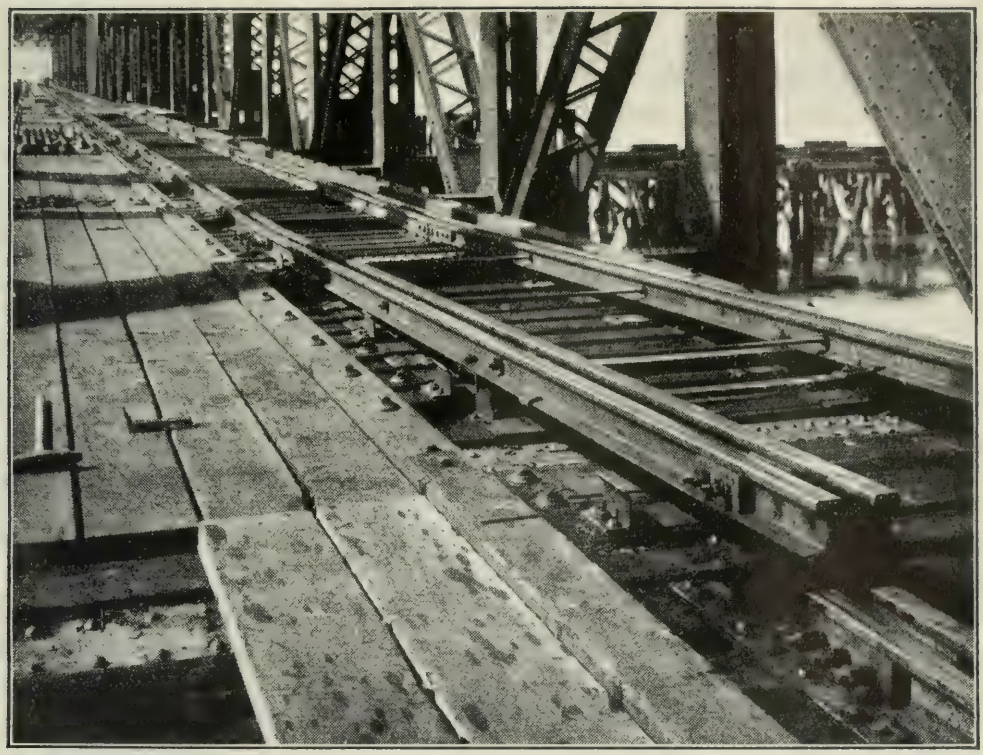


FIG. 34.—LIFT RAILS, DUMBARTON POINT BRIDGE.

FIG. 35.—DUMBARTON POINT BRIDGE.

site the barges were placed under the spans at low water, so that the rising tide would lift the steelwork clear from the temporary pile supports. The spans were towed against the tide (Fig. 29). When a span got into position between its cylinder piers (Fig. 30) block and tackle lines were thrown out and made fast to the piers. These lines were connected with donkey engines and capstans, so that a span could be adjusted to its proper position over the grillage beams (Fig. 31) and hold it while being lowered. The lowering was accomplished by opening sea valves in the barge. When the span was in place on the grillage, the cribbing was knocked out and the barge withdrawn. The average time required to place a span, from the time it left the falsework on the shore until the removal of the barge from under it at the piers, was about 2 hours, but in each case depended somewhat on the distance the span had to be towed.

To provide for a continuous rail (Fig. 34) at the ends of the draw span, the inside flanges of the guide and traffic rails were cut off flush with the web to permit the rails to slip over a cast-steel filler block. The rails were hinged with a slotted hole at the other end, where a similar casting was placed. The engine which provides turning power for the draw also operates the end track-lifting devices. The bridge is opened or closed in 2 min. Fig 35 is a view of the completed bridge.

About 15 000 cu. yd. of concrete were placed in the foundations; 3 137 cu. yd. were required for the pivot pier. The total weight of steel is 5 935 tons, divided about as follows: draw span, including machinery, 1 215 tons; center pier, 250 tons; six 180-ft. approach spans, 480 tons each; in all approach and rest piers, 1 590 tons.

The work was designed and constructed under the supervision of William Hood, M. Am. Soc. C. E., Chief Engineer, Southern Pacific Company, to whom the writer is indebted for much of the information contained in this paper. The field work was in charge of Mr. W. E. Marsh, Assistant Engineer, until his death, in September, 1909. Too much credit cannot be given to Mr. Marsh for devising with success the many field details and operations which were developed as the work progressed. Mr. C. R. Broughton, his assistant, then took charge of the field work, and to him and to Mr. C. M. Kurtz, who took the photographs, the writer is under obligations for assistance in collecting the facts described in this paper.

DISCUSSION

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is very much interested in the subject presented in this paper, inasmuch as he was officially connected with the progressive changes made in the proposed plans, and in the final acceptance of the bridge after completion. The local struggles with bad foundations make the construction of the "Dumbarton Cut-off" one of the most instructive pieces of bridgework in the State of California.

The first trouble encountered was when an attempt was made to put in ordinary trestle approaches from the shore line on each side out to the site of the draw span. These approaches, as well as the draw span, were double-track structures. After a long and expensive struggle, it was found to be impossible to maintain a trestle bridge in the deep and strong tidal currents which prevail at the bridge site. It was finally concluded to put in six 180-ft. through riveted spans, three on each side of the draw-span. This caused a great increase in the cost, which was naturally very embarrassing.

The troubles with bad foundations, however, had just begun to develop. For the land approach across the salt marsh from Newark, westward to the bay shore, the plan adopted was to build the embankment by dredging two canals (one on each side), and depositing the material in the embankment lying between. This method led to much trouble in the maintenance of way. The first tracks laid, of course, were only for construction trains, but the embankment settled so irregularly that the track lines could not be kept passable, even for them, which means a great deal to one of experience. This trouble continued during the time of the construction of the steel bridge in its entirety. The track lines could not be maintained in serviceable condition, because of the settlement of the embankment and the gradual rising of the beds of the two canals on each side. Finally, the Southern Pacific Company, in desperation, obtained two hydraulic dredges, and put them to work filling up the two canals with good heavy materials, dredged from the bay shore and deposited through two long pipe lines. This expedient had the desired result, and settlement of the embankment practically came to an end. The permanent track lines were then established and well ballasted, and the "Dumbarton Cut-off", after three years' operations, was officially opened to general traffic on September 30th, 1910.

The author's description of the method of putting in the center pier of the draw-span is most interesting and instructive. Of course, engineers differ as to the best method of laying concrete under water. The writer is decidedly in favor of the judicious use of the "tremie", as being the most rapid, and therefore the most satisfactory, method. The principle to bear constantly in mind is that, where the face of the

freshly laid concrete is being constantly washed by the ponded water, the faster the concrete is laid the less effect the water will have on it. By rushing the rate of laying, the water has not time enough to act on any one face before a new layer is right on top of it, hence, the quicker the laying is done, the better the results. Mr. Le Conte.

This principle was first called to the writer's attention in 1872, by the late Gen. B. S. Alexander, Corps of Engineers, U. S. A., who stated that he got it from the late Gen. Joseph Gilbert Totten, Corps of Engineers, U. S. A., Hon. M. Am. Soc. C. E., at Dry Tortugas, off the south end of Florida. The experience, therefore, is of long standing.

The writer wishes to express his profound regards for the late Mr. W. E. Marsh, Assistant Engineer, Southern Pacific Company, who extended many courtesies to him during his official inspections. The Dumbarton Bridge stands to-day as a lasting monument to wise judgment and good construction.

R. D. Coombs, M. Am. Soc. C. E.—The speaker is particularly impressed with the size of the piles used, the lengths being from 60 to 120 ft. and the diameter of the tips either 9 or 10 in. In the East such sizes are not obtainable for ordinary work. The question arises: Was it thought necessary to use large tips in order to increase the bearing power of the piles, a necessity not indicated, at least to the speaker's mind, if the underlying soil was sand and clay? Mr. Coombs.

A certain proportion of the foundation piles extended up into the concrete piers, presumably as an anchorage, and the speaker would inquire whether such piles were peeled. In several cases in which the concrete foundations were very shallow, the speaker has extended the piles as far as possible into the concrete, using peeled piles with spikes driven into the sides of the embedded portion. The exact anchorage value of this method is not definitely known, but it is undoubtedly considerable.

Referring to the advisability of penetrating so-called crusts, the speaker believes it is usually necessary to do so in pile work, particularly where the "crust" is a vegetable deposit underlaid by sand or clay, or both. In some sections of the New Jersey swamps there is a peat-like deposit about 10 ft. deep, which, if unbroken, might carry a highway load, but is useless for pile foundations. However, the sand and clay below will support heavy loads by skin friction very effectually.

Is it not true that the old formula for the drop-hammer is just as inaccurate as any new formula for the steam-hammer? Is not pile-driving mainly a matter of judgment? In the speaker's opinion, it is.

As an illustration, two more or less typical experiences may be cited: In one, the foreman informed the speaker that he had just driven a spliced pile 190 ft. long. Naturally, he was at once assured

Mr. Coombs. that he had not done anything of the kind. The fact was that, through a conscientious attempt to obtain the usual penetration per blow, the piles were being broken at the splices and driven horizontally. Hard bottom was not more than 40 ft. below the surface.

In the second case, piles from 65 to 75 ft. long, driven in blue clay, for a wharf, were stopped under easy driving to prevent them from going below cut-off. A few months later, stevedores unloaded 80-lb. rails on the wharf, piling them 6 ft. high, and there was no settlement.

Mr. Hovey. O. E. Hovey, M. Am. Soc. C. E.—The speaker has been much interested in this concise and well-illustrated paper; and, though there are many interesting details in the draw-bridge superstructure which might be mentioned if any extended discussion of it was undertaken, only a few points, which were of particular interest to him during the design of the draw-span, early in 1907, will be mentioned. The Dumbarton draw-span was one of the heaviest center-bearing spans which had come to the speaker's notice at the time the design was made. The center-bearing type of draw was considered to be particularly desirable in this location on account of the difficulty in securing foundations, the simpler details of the center, the rigidity of the bridge under traffic after the wedges at its ends and center were driven home, and the ease with which final adjustments of height of center and end supports could be made during construction in case there was any settlement of the piers. The center bearing of this bridge was designed for a total load of about 2 600 000 lb. Grillages of I-beams were embedded in the concrete of the end and center piers, under the end and center wedge supports and the center pivot. The center pivot casting rested directly on the top of the center grillage, and the disks at the top of this casting consisted of two steel plano-concave and one phosphor-bronze double convex disks, 36 in. in diameter. These disks were the largest with which the speaker was familiar at the time the design was made, although much larger ones have been used more recently in the manufacture of draw-bridges, the largest with which the speaker is familiar being those supporting the Southern Pacific draw-span over the Sacramento River, at Sacramento, Cal., which are 52 in. in diameter.

The design of the power plant in the operator's house of the Dumbarton draw is of interest because, although it was expected ultimately to use electric energy for operating the draw, electric current would not be available at the time the bridge was completed, and it was necessary, therefore, to provide some other source of power temporarily, and desirable to have an alternate power available, even after the electric current was in use. For the permanent equipment a 52-h.p. motor, designed to use 440-volt alternating current, was provided. For temporary and alternative use an 80-h.p., three-cylinder, four-cycle, gaso-

line engine, running at about 300 rev. per min., was selected. In the operation of this draw-bridge it is necessary to provide, from one source of energy, means to lower and raise the center and end wedges, and also to open and close the bridge itself, four separate operations being required. The type of gasoline engine selected was not reversible, therefore special transmission mechanism was necessary to produce rotations in each direction for lowering and raising the ends of the bridge and for opening and closing the main structure. Many bridges have been built with mechanism to provide for these four motions, but, in a large number of cases, the necessary shafts, gears, clutches, levers, etc., have been located in various positions on the superstructure. In conformity with the well-established practice of the builders of this bridge, it was decided to use a type of transmission mechanism, between the gasoline engine and the main shafts of the bridge, which would be entirely within the operator's house and self-contained on a large cast-iron bed-plate, so that the transmission mechanism could be assembled and carefully aligned in the shop and then shipped to the bridge in such condition that a minimum of field work would be necessary in placing and adjusting it. This transmission mechanism included two mechanical and two friction clutches, with the necessary shafts, gears, and friction-band brake control, all actuated by the operator from a central point. Mr. Hovey

The electric motor selected was of the ordinary reversible type, and therefore required special transmission mechanism to shift the application of power to the wedge-operating shafts or bridge-turning shafts at pleasure; but it did not require special mechanical provision for reversing the direction of rotation. To keep the whole mechanism compact and rigid, it was decided to place the motor on a cast-iron base which was carefully fitted and bolted against one end of the cast-iron base of the gasoline engine transmission mechanism. The motor base also supported bearings for a countershaft, the pinion of which was arranged to mesh directly with a gear on the extended end of a countershaft of the gasoline engine transmission mechanism. The motor countershaft was also provided with a friction cut-off coupling, so that by moving a hand-lever the motor could be thrown into engagement with the gasoline engine transmission mechanism or allowed to run idle. By this construction, it will be noted, the transmission mechanism can be driven by the gasoline engine or by the electric motor, at pleasure, and in case of necessity a shift from electric power to gasoline engine power can be made simply by releasing a clutch and starting the gasoline engine.

On account of the location and weight of the bridge, the time of opening and closing was fixed at 2 min., as stated in the paper. During a visit to the Pacific Coast in July, 1912, the speaker had an oppor-

Mr. Hovey.

tunity to inspect the Dumbarton Draw-Bridge, and during this inspection the bridge was opened by the operator, who did not know that he was being timed. Beginning with the first movement to set the railroad signals—the signals were set at danger—the operator on the bridge received his signal to open, and the cold gasoline engine was started and operating speed was attained in 60 sec.; 20 sec. were required to lower the ends of the bridge; 5 sec. were used in throwing the clutches from the wedge-operating shafts to the bridge-turning shafts; and 85 sec. sufficed to open the bridge 90°; thus making the total time from the first motion of the signals to the fully open position of the bridge, 2 min. and 50 sec.

The speaker visited many draw-bridges on the Pacific Coast, but did not see an operator's house, or a set of bridge mechanism, in such perfect condition as that on the Dumbarton Bridge. The bridge operator evidently took great pride in the maintenance of the structure. The bridge machinery, after practically two years' use, was as clean as one would expect to find well-maintained machines in a shop, and the operator's house, in all its appointments, was a model of neatness.

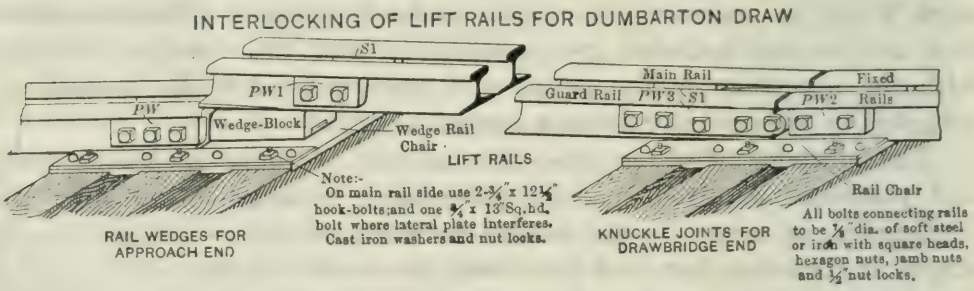

INTERLOCKING OF LIFT RAILS FOR DUMBARTON DRAW
S1
PW1
PW
Wedge-Block
Wedge Rail Chair
LIFT RAILS
Main Rail
Guard Rail
PW3
S1
PW2
Fixed Rails
Note:-
On main rail side use 2-¾"x 12½" hook-bolts;and one ¾"x 13"Sq.hd. bolt where lateral plate interferes. Cast iron washers and nut locks.
Rail Chair
All bolts connecting rails to be ⅛"dia. of soft steel or iron with square heads, hexagon nuts, jamb nuts and ½"nut locks.
RAIL WEDGES FOR APPROACH END
KNUCKLE JOINTS FOR DRAWBRIDGE END

FIG. 36.

The author mentions the lifting rail joints, but does not give full details of their construction. The speaker observed that these joints were in perfect condition, after having been in service for about two years, under heavy freight traffic, and that they were of a somewhat unusual design, their construction being illustrated in Fig. 36.

The main and guard-lifting rails are bolted together with close-fitting separators, *S* 1, hinged to the fixed rails on the draw-span, and supported on the rail chair, *R C*. The lifting ends extend over on to the approach span, a distance of about 4 ft., where they seat on a wedge rail chair, *W R C*. The bottom flanges of the main and guard rails are coped off on the inside to allow them to slide up and down on the wedge-blocks which are riveted down to the base-plates of the wedge rail chair by countersunk rivets, and form a firm support, holding the rails true to gauge.

The wedge-block is of such shape as to clear the flanges of passing wheels, and its sides are extended up and finished to fit the under side of the rail heads, thus giving them good support. In a later design,

short easing rails were added outside of the main rails to support wheels with worn treads while passing over the joint, so that their flanges would not rest on the wedge-blocks. Lifting rails of many designs have proved unsatisfactory and expensive to maintain and operate, and experience with funnel rails seems to justify their use. Mr. Hovey.

In reference to the large piles used on this work, the speaker would state that it is much easier and cheaper to obtain such piles on the Pacific Coast than near New York City, and their use insures a better foundation than is likely to be obtained by the small slender piles frequently found in this section of the country.

Lewis D. Rights, M. Am. Soc. C. E.—The author is to be congratulated on the concise and clear manner in which he has covered the engineering features connected with this bridge. The speaker is interested in the description of the erection of the steelwork, which seems to have proceeded in a very expeditious manner. Mr. Rights.

Mr. Schneider speaks of the novel construction of a derrick-car, which, according to the illustrations, consisted of a hinged **A**-frame placed on a standard flatcar. Such tools have become common in the East, and almost every bridge company owns one or more. Hinged **A**-frames of wood have been in use for some years, and, about ten years ago, S. P. Mitchell, M. Am. Soc. C. E., conceived the idea of building a special steel car with a hinged steel **A**-frame. A car of this kind was used successfully in connection with the erection of the Warehouse Point Bridge, where the New York, New Haven and Hartford Railroad crosses the Connecticut River, below Springfield, Mass. Mr. Mitchell then substituted heavy steel struts for back-stays, patented the invention, and has built, or authorized the construction of, a number of cars. The **A**-frame is 23 ft. high, making the total height of the car when in use about 27 ft. Such height of mast is necessary for booms of 50 and 60 ft., and cars have been built recently with masts some 7 ft. higher. The standard overhead clearance for the Middle Western roads is 22 ft. above the rail, and as this is not sufficient to clear the top of the **A**-frame, Mr. Mitchell has made arrangements to lean the frame back and shorten the back-struts by a series of pin-holes in the latter. On some of the Eastern railroads, especially the New England roads, the overhead clearance is very much less than 22 ft.; in some cases it is only about 15 ft., with a minimum of 14 ft. 6 in. This is not much more than sufficient to pass a high furniture car, and it is not always practicable to lean the **A**-frame back far enough to be able to work under such clearances. In the last few years some of the Eastern concerns have attempted to overcome this difficulty by building a heavy steel frame, the top chords of which occupy about the same position as the eaves of a box car. One company has made the front posts of this frame hollow, so that

Mr. Rights

an A-frame can telescope into them. An arrangement of pin-holes enables work to be done with an A-frame at a height very little above that of a box car, or to extend the frame so as to give a mast of about 18 ft. Several other companies have used this same type, but have hinged a short A-frame on the top of the steel truss, instead of at the car floor.

Another feature of interest in connection with the erection of the Dumbarton Bridge is the illustration of the double-headed traveler. It would seem that the single booms at each end of this traveler were designed with the idea of working alternately on the fixed spans, and that probably only one boom was worked to advantage at any one time. A single boom for the erection of a truss was not considered good practice several years ago, owing to the number of pieces which had to be held in place while a connection was being made, but the introduction of riveted joints has produced quite a change in methods. Recently, derrick-cars have been used very successfully in the erection of riveted trusses, although there are engineers and erectors who maintain that other tools are more efficient. In this matter, it is sometimes difficult to decide as to the most efficient method of erecting steelwork. Several years ago, the New York, New Haven and Hartford Railroad, in extending its six-track line into New York City, built a great number of overhead bridges for the separation of street and railroad grades. The bridges were designed for New York City paving standards and very heavy live loads, and consisted of three trusses with double roadways and sidewalks. The spans were not very long, 120 ft. probably being the average. It happened that during a period of about 18 months, four different bridge companies secured contracts for four bridges, the details of which were very similar. They were all overhead highway bridges, with the erection conditions almost exactly the same, and the interesting feature was that each of the four companies used a different method of erection.

One company used a steel gallows, or gantry traveler, reaching above the trusses, and erected the steel from fixed hoists.

The second company erected two heavy steel derricks in the street, one at each abutment, with booms long enough to meet at the middle of the span.

The third company had two derrick cars, one of which was placed on the upper falsework and was used for erecting the trusses; the other was used down on the railroad grade to supply the upper one with material.

The fourth concern built a two-boom mule traveler, that is, a low-decked traveler with two masts and booms. The booms reached out over the falsework to get material from the cars below.

Mr. Rights.

The speaker was interested in the erection of one lot of bridges, and watched carefully to see what the other companies were doing, in order to learn whether there was any marked difference in the methods. As far as he could see, each concern was guided in the choice of erection tools by what it had on hand, and there did not seem to be any very great advantage of one kind of tool over the other. The speaker is satisfied that the method in which he was interested was the best; but the representatives of the other concerns feel the same way about their methods, so that the question is still open.

Mr. Belzner.

THEODORE BELZNER, ASSOC. AM. SOC. C. E. (by letter).—The floating of the six approach spans into position, recalls to the writer operations of a similar nature on the Harlem River Ship Canal Bridge during the fall of 1905 and the summer of 1906.

The author states that the average time required to place a span was about 2 hours, depending on the distance it had to be moved. More information concerning the itinerary of the work, such as rate of motion, distance floated, etc., for each operation, would be of interest.

In reference to the floating of the approach and draw-spans of the Harlem River Ship Canal Bridge at Kingsbridge, it may be of interest to state briefly the actual times required in the six operations of moving the old and placing the new spans.

The Department of Bridges of the City of New York issued a permit closing the Ship Canal Bridge to street travel for a total time of 5 days, of which not more than 3 days were to be consecutive. It was calculated that only 1 day would be necessary for moving each of the approach spans and that the draw-span operations could be completed in 3 days.

Prior to the removal of the old approach spans, the new spans were moved to points about 115 ft. east and west of their final positions (these movements consisting of a land and water operation). The two old spans were 100 ft. long and weighed 240 tons each; the two new spans were 100 and 112 ft. long, and weighed 330 and 380 tons, respectively. The old draw-span was 272 ft. long, weighed 900 tons, exclusive of cribbing, and was floated from Kingsbridge to the University Heights site, a distance of about 1 mile. The new draw-span was also 272 ft. long, weighed 1 500 tons (and with the cribbing nearly 2 000 tons), and was floated from 216th Street, a distance of $\frac{1}{2}$ mile to its new site at Kingsbridge.

On October 19th, 1905, at 5 A. M., the south approach span operation was begun, and at 5.45 P. M. the first vehicle passed over the bridge, the time elapsed being $12\frac{3}{4}$ hours. On November 6th, at 1.30 P. M., the north approach operation was begun, and at 10.15 P. M. the bridge was opened to surface traffic, the time elapsed being $8\frac{3}{4}$ hours. These

Mr. Belzner.

operations delayed travel 21½ hours, or 26½ hours less than the calculated time.

The draw-span operations extended from 10.15 A. M. on June 14th, 1906, to 5.20 A. M. on June 17th, the time elapsed being 2 days, 19 hours, and 5 min., or 4 hours and 55 min. less than the stipulated time.

In other words, the entire operations were accomplished in 3 days, 16 hours and 35 min., or 1 day, 7 hours and 25 min. less than the 5-day limit.

Mr. Schneider.

E. J. SCHNEIDER, M. AM. SOC. C. E. (by letter).—It has been particularly gratifying to the writer to note the interest which has been taken in this paper. Certain points are raised by Mr. Rights regarding the double-ended traveler: The single booms at each end of the traveler were designed especially for facility in erection of the draw-span, which was erected as symmetrically as practicable, from the center toward each end, on top of the protection work which had been designed to act as falsework. Had the traveler not been double-ended, additional falsework would have been required at one end of the draw-span protection and at the site of the erection of the six 180-ft. spans.

Answering Mr. Coombs in regard to piles and pile-driving: The specifications called for lengths of from 60 to 120 ft., the diameter of the tips to be 9 or 10 in., the usual requirements by the Southern Pacific Company for ordinary work, and no change was made in them for this particular piece of construction. Pacific Coast piles, no doubt, are larger than those ordinarily used in the East. All piles used at Dumbarton were peeled.

TABLE 1.—RECORD FOR FLOATING SIX 180-FOOT SPANS FROM FALSEWORK TO PIERS, FOR THE DUMBARTON BRIDGE.

The Span Numbers Run from East to West, Numbering the Entire Draw-Bridge as Span No. 4.

Date.	Span No.	Hour.		Time interval.	Approximate distance floated, in feet.
		Clear of falsework.	Landed on piers.		
September 16th, 1909.....	3	12.25 P. M.	2.00 P. M.	1 hr. 35 min.	700
September 26th, 1909.....	2	9.30 A. M.	11.00 A. M.	1 " 30 "	500
October 14th, 1909.....	1	10.55 A. M.	12.15 P. M.	1 " 20 "	300
November 7th, 1909.....	5	8.25 A. M.	9.40 A. M.	1 " 15 "	1 200
December 9th, 1909.....	6	7.45 A. M.	9.50 A. M.	2 " 05 "	1 400
December 11th, 1909.....	7	10.05 A. M.	11.05 A. M.	2 " 00 "	1 600

Pile-driving is mainly a matter of judgment. The lengths of the piles at Dumbarton were determined from actual test driving on the

Mr. Schneider.

line across the Bay, and from the records of the borings made by the Spring Valley Water Company on the line of its submerged pipe a short distance from, and approximately parallel to, the bridge.

The points brought up by Mr. Belzner, in reference to the average rate of motion, distance floated, etc., for each operation, can best be answered by a copy of the record for floating the six spans from the falsework to the piers, Table 1.

The average rate of motion for each operation was a little less than 1 mile per hour.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1272

THE ABSORPTION OF OXYGEN BY DE-AERATED WATER.

By Earle B. Phelps, Assoc. Am. Soc. C. E.

With Discussion by Messrs. W. E. Adeney and Earle B. Phelps.

The Report of the Metropolitan Sewerage Commission of New York, dated August 1st, 1912, and recently issued, contains, among other interesting material, a Report by Dr. W. E. Adeney, in which especial attention is given to the subject of re-aeration of polluted tidal waters. Dr. Adeney's views and conclusions are in many respects directly contradictory to those which William M. Black, M. Am. Soc. C. E., Colonel, Corps of Engineers, United States Army, and the writer reached after a study of this problem, in connection with New York Harbor, and fully stated in a report to the Board of Estimate and Apportionment of New York City, published March 23d, 1911. As this last-named report did not have a wide circulation, and as the matter under discussion is one of great importance, not only in the case of New York Harbor, but in all similar situations, it seems appropriate at this time to consider the facts and the conclusions that have been drawn therefrom by various workers.

Dr. Adeney made the first important contribution to this subject in his well-known experiments undertaken on behalf of the British Royal Commission on Sewage Disposal. His results are fully set forth in Appendix VI of the Fifth Report of that Commission.

He describes experiments, with glass tubes 6 ft. long and $\frac{3}{4}$ in. in internal diameter, in which he exposed de-aerated water under varying conditions, and, at the expiration of stated times, withdrew samples

from various depths and submitted them to analysis for dissolved oxygen and nitrogen. Both sea water and distilled water were used. He found that, with either kind of water, mere exposure of the upper surface to the air resulted in practically negligible re-aeration in the lower depths. He reported a tendency toward accumulation of the dissolved gases in the upper layers, which was greater in the distilled water than in the sea water, under these conditions. He next closed the top of the tube, introduced a small tube just below the surface of the water, and, by suction, caused a slight current of air to bubble through the top layer of water. This led to a slightly greater absorption on the part of the distilled water. In the case of the sea water, however, this method of procedure resulted in a fairly rapid absorption of oxygen and of nitrogen, and the quantities of these gases found at various depths at the conclusion of each experiment showed a tendency toward uniform distribution.

Dr. Adeney refers to this phenomenon as the "streaming effect", and identifies it with a similar effect obtained by Huefner under quite different circumstances. He adduces evidence, although inconclusive, that Huefner's theory of a direct streaming of the liquid, brought about by difference in specific gravity, is not tenable. He did not investigate fully the cause of the streaming effect, but believed it might be due to:

"* * * minute dust particles, or by other centres of condensation, possibly of an electrical nature, carried by the air current and being taken up by the water together with the gaseous constituents of the air, and that these bring about in some way a sufficient density of the dissolved or condensed gases to render it possible for them to be drawn gravitationally downwards through water."*

The previous passage of the air through one tube of water destroyed to a large degree its power to bring about this streaming effect in a second tube, while the power was completely lost by passing it twice through water, thence into a third tube. Dr. Adeney suggested that the ions, or dust particles, or whatever may be the exciting cause, were removed by such treatment.

Col. Black and the writer showed by experiments with dyes and by determining the amount of evaporation resulting from the passage of dry, or partly dry, air into a tube of water, following Dr. Adeney's

*British Royal Commission on Sewage Disposal, Fifth Report, Appendix VI, p. 64.

technique, that this streaming effect, so-called, was caused by a true streaming of water currents. These currents are caused by an increased specific gravity of the salt solution following evaporation of the water from the surface layer. This explains also the effect of using two or three tubes in series, and the absence of any material streaming in fresh water. In the latter case, a slight tendency toward streaming was observed, and, without further investigation, this was ascribed to a slight lowering of the temperature at the surface, due to evaporation, and the consequent increase in the density of the surface layer. Undoubtedly, also, the increased density resulting from the solution of atmospheric gases is a factor, as shown by Huefner. Although Dr. Adeney in his first report claimed to have disproven the theory of circulating currents of water, the explanation of this matter by Col. Black and the writer has been tacitly accepted by him in his report to the New York Commission, so that there need be no further discussion of the theory involved. The real point at issue is indicated in the following statements:*

"It is equally evident from these experiments that whatever may subsequently be discovered to be the true cause of the streaming, its effect in large volumes of sea or river water under natural conditions must be of great importance, and of such dimensions that the effect of ordinary diffusion may, in comparison, be entirely neglected."

Col. Black and the writer state:†

"In practice no such results would be observed, since the upper layers of an estuary are always somewhat fresher than the lower."

This latter statement, Dr. Adeney says, has no real foundation in fact. In this connection, reference to the New York Commission's report‡ will be of interest. Here is shown the condition of the water at The Narrows, as regards both dissolved oxygen and salinity or percentage of sea water, at seven different stages of the complete tidal cycle. A distinct stratification is shown in each instance. From the surface to the 20-ft. depth there is an average increase of about 6% of sea water. In the case which shows the greatest uniformity throughout the depth, there is more than 2% increase. Moreover, the condition at the Narrows is less favorable to the contention of Col. Black and the writer than the majority of the cases shown in the

* British Royal Commission on Sewage Disposal, Fifth Report, Appendix VI, p. 64.

† Report of Col. W. M. Black and Prof. Earle B. Phelps concerning The Location of Sewer Outlets and The Discharge of Sewage into New York Harbor, 1911, p. 52.

‡ Metropolitan Sewerage Commission of New York, 1912, p. 449.

diagrams which follow. At the mouth of the Hudson, the gradient ranges from 6 to 8% of sea water in 20 ft., and in the upper Hudson, at Mount St. Vincent, it is normally more than 10% in the same vertical distance.

There seems to be, therefore, sufficient "real foundation in fact" for the assertion that stratification does exist, and that there is, therefore, no such vertical circulation of water from top to bottom, as Dr. Adeney found in his tubes, which were initially at a uniform concentration. If further foundation in fact be required for this almost self-evident proposition, reference may be had to the work of Morton F. Sanborn, Assoc. M. Am. Soc. C. E., on "The Distribution of Sea Water in the Charles River Basin."* As a result of a year's study, with almost daily examination of the vertical distribution of sea water in various parts of the Charles River Basin at Boston, Mr. Sanborn found stratification to be so persistent that the most severe storms produced only a temporary uniformity, even in the upper 20 ft. of the water, and that, after the cessation of such storms, stratification was re-established. The analytical evidence is direct and positive, and rests on no theory. The streaming effect which depends on the presence of more dense layers above those of less density, is impossible in waters that show increasing concentration of salt with increasing depths. These conditions exist in the cases that have been cited; therefore, the phenomenon of streaming does not and cannot occur.

Dr. Adeney's further suggestion that the sewage matters are concentrated in the upper layers is obviously true, but beside the question. We are considering the rate of absorption of dissolved oxygen by a polluted water, under the restriction that the dissolved oxygen content shall not be reduced below a certain specified amount. This value, if properly determined, will limit the amount of sewage which may be received into the water in question, under the restrictions imposed. The most favorable assumption that can be made is that the sewage is uniformly distributed. This, in other words, will give a maximum permissible pollution. If, on the other hand, we assume a concentration of pollution in the upper layers, then either the amount of permissible pollution will be decreased in proportion to this decreased volume of water available, or the conditions at the surface will be reduced below the assumed standard. Such a concentration of

* *Engineering News*, March 10th, 1910, p. 272.

sewage near the surface would result in a greater average rate of oxidation than if there were uniform distribution of the same quantity of sewage, but it is not the condition of maximum re-aeration.

Dr. Adeney next refers to the experimental technique by which Col. Black and the writer determined the diffusion coefficient. He first points out that that coefficient bears no resemblance to the one determined by Huefner, and believes that our series of ascending values with increasing temperature are due to a streaming effect, because of failure to take extreme precaution in preserving the purity of the water used. With regard to the first point, the apparent discrepancy is merely one of units used in the formula. The exact definition of a complicated coefficient of this kind cannot be simply made, and the reader is referred to the original paper (page 89) for the definition of the coefficient and the units in which it is stated. It would be a laborious matter to convert Huefner's coefficient to these units, and the magnitude of the experimental work was so much greater than his, and, in the writer's estimation, the method of experimenting was so much more reliable, that no attempt has been made to recompute the results in other terms. The form in which Col. Black and the writer have derived it is most favorable to practical application to the problem in hand.

As for the general accuracy of the analytical work, the results must speak for themselves. It would seem that Dr. Adeney's statement, "That the results obtained by these observers do not represent the true co-efficient of diffusion of oxygen in water becomes evident on carefully examining them" (page 83), and his further statement that these results have "no resemblance nor relationship to the true co-efficient," might have been accompanied with more explanatory matter for the benefit of those less familiar with the problem. Col. Black and the writer in their report give a plot, Fig. 1, showing the results of 114 tests made throughout a considerable range of temperature. Dr. Adeney quotes from page 94 of that report as follows:

"The values are somewhat scattering, and a closer analysis indicates that some unknown factor is involved. Exceedingly concordant results are readily obtained upon any one day, while upon the following day another group of concordant results will be obtained differing from the first. Barometric corrections were later applied without relieving the situation."

From this statement, Dr. Adeney concludes that our values have been arbitrarily selected, and will give misleading results. The plot, Fig. 1, shows the general character of the errors that have been referred to. There are admittedly imperfections in the process, and the

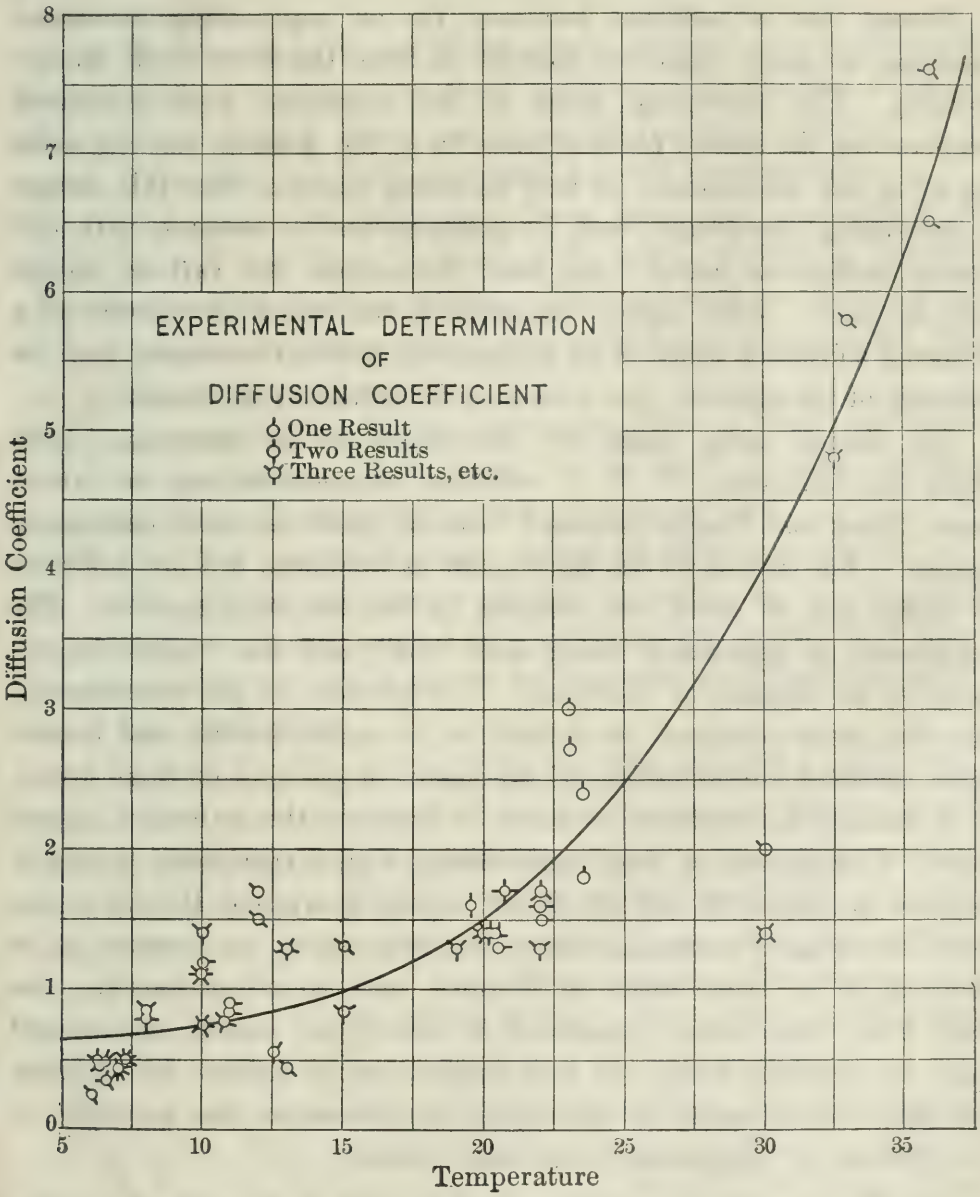


FIG. 1.

results are not of extreme scientific accuracy. For this reason, the work, which was originally carried out as a contribution to pure science, was not published as such. On the other hand, as stated on page 95:

"Within the range of values which have been employed in the computations in the text, errors resulting from the maximum divergence of individual tests from the average value of the curve are less than 5 per cent. at the five-foot depths and less than 2 per cent. at the twenty-foot depths."

Surely this is sufficient accuracy for an engineering discussion involving so many unknown factors, as does the New York Harbor problem. The increasing value of the coefficient with increasing temperature, the second point objected to by Dr. Adeney, was not made use of in our discussions. It may be noted, however, that this change is thoroughly concordant, both in direction and in amount, with diffusion coefficients which have been determined for various soluble salts in water. With higher temperature, the molecular velocity of a dissolved substance (that is its tendency to diffuse) increases, and the viscosity of the solution (the resistance to diffusion) decreases.

Dr. Adeney gives (page 85) the results of an experiment from which he concludes: "* * * whether evaporation can or cannot occur, Black and Phelps' proposed formula yields seriously misleading results." The details of the experiment as published are not sufficient to enable one to check our formula by definite computations. The experiment is apparently made with "air", and the results are referred to as "Degree of Aeration." The portions of the experimental tube that were examined are defined as the upper, middle, and bottom layers, without any definition of the extent or position of these layers. It is impossible, therefore, to prove or disprove the statement quoted above. Comparison of these experiments with Experiments 8 and 9, reported on pages 62 and 63 of Dr. Adeney's original Report to the Royal Sewerage Commission shows the two sets of experiments to be identical in all their essential features, and, in all probability, the New York experiments numbered 3 and 4 are merely recomputed from the original work. If this supposition is correct, Dr. Adeney has made seven errors in attempting to determine the accuracy of our formula by comparison with these results:

(1) The formula is misinterpreted in that it is assumed to give the increased oxygen content at any depth, whereas its derivation and our use of it show that it gives the total oxygen absorbed by a column of stated depth. This is plainly stated on page 91 of our report.

(2) Our 20° coefficient is used, although the temperature of the

experiments is about 8.5 degrees. Our coefficient for that temperature is just half the coefficient at 20 degrees.

(3) The "re-aeration" was measured by nitrogen, while our coefficient refers to oxygen. There is no evidence that the two gases behave similarly.

(4) The experiments are in sea water, whereas our coefficient refers to fresh water, and we state explicitly that the sea water coefficient is necessarily lower, owing to the increased viscosity of sea water. This effect would be exaggerated at low temperatures.

(5) The computed value was based upon the supposition that the water was free from oxygen at the start, whereas it contained more than 16% in each experiment. Proper use of our formula takes account of the original oxygen content, a self-evident matter which Dr. Adeney consistently overlooks.

(6) In recomputing his results to the basis of "per cent. saturation," the temperature of the room, 10° to 13.5°, is used, instead of the temperature of the water jacket, 8.5 degrees.

(7) In these recomputations, values of zero are arrived at in the middle and bottom layers, whereas in each case in the original experiments a definite amount of re-aeration was noted. The quantities are small, but too large to be ignored when the argument is based upon zero values.

If the writer is in error in assuming that these two sets of results refer to the same experiments, it is merely necessary to point out that they are obtained under identical conditions, so far as the conditions are stated, and, in any event, the writer will deal with the original work, which is fully described, rather than with the apparently recomputed values of the New York Report. The figures of the original report, of the New York Report, and of the former recomputed to the form of the latter, are given in Tables 1 and 2.

The columns referred to in Tables 1 and 2 are the long glass tubes. Air was bubbled through the top layers of three of these in series. The results of the first columns are not given, for the circulating currents established vitiated the diffusion effect.

The experiments are not capable of giving an accurate measure of the diffusion coefficient because of the large percentage errors that are possible and evident in the lower depths of all the columns. Neither are they planned to take advantage of our formula, which gives the

TABLE 1.—DR. ADENEY'S EXPERIMENTS WITH SEA WATER.
Original Experiments. Royal Sewerage Commission Report, Pages 62–63.
Amount of nitrogen in cubic centimeters per liter.

No.	Time, in hours.	Temperature, Centigrade.		1 to 200 mm.		800 to 1 000 mm.		1 600 to 1 800 mm.	
				Columns:		Columns:		Columns:	
		Room.	Jacket.	2	3	2	3	2	3
8.....	Start.	12.8° to 10.0°	8.6° to 8.3°	2.13	2.51	2.13	2.51	2.13	2.51
	48			11.15	11.09	2.29	2.55	2.18	2.54
9.....	Start.	13.7° to 13.5°	8.4° to 8.5°	1.63	1.51	1.63	1.51	1.63	1.51
	48			7.15	6.93	1.65	1.56	1.65	1.52

TABLE 2.—DEGREE OF AERATION, AS PERCENTAGE OF SATURATION.

No.	Time, in hours.	Temperature, centigrade.	Uppermost Layer.		Middle Layer.		Bottommost Layer.	
			Columns:		Columns:		Columns:	
			2	3	2	3	2	3
Computations in New York Report, Page 85.								
3.....	48	10.0	72	69	0	0	0	0
4.....	48	13.5	47	47	0	0	0	0
Computations by the Writer for Nitrogen at 8.5 Degrees.								
8.....	Increase in N in 48 hours (in cubic centimeters)		9.02	8.58	0.10	0.04	0.05	0.03
	" " " (percentage of saturation).....		70	67	1.24	0.31	0.39	0.23
9.....	Increase in N in 48 hours (in cubic centimeters)		5.52	5.42	0.02	0.05	0.02	0.01
	" " " (percentage of saturation).....		43	42	0.16	0.39	0.16	0.08

total oxygen absorbed in the entire column. In the course of establishing the formula, we derived an expression for the change in oxygen concentration at any depth. This is shown on page 92, just before the integration. While we have no determination of the coefficient for nitrogen in sea water, it may not be without interest to note that by this formula, under the stated conditions of depth, temperature, and initial content of the water, the increase in oxygen in a column of fresh water at the bottom layer, depth, 1 700 mm., would amount to but 0.1% of saturation, instead of 6%, which Dr. Adeney computes. In Experiment 9, in which the temperature was maintained constant

within 0.1°, the results were 0.16% and 0.08% in the two tables. In the other experiment the temperature varied 0.3°, and the results were 0.39% and 0.23%, respectively. Undoubtedly, all results are high, owing to changing temperature and circulation in the tubes. It is evident, however, that the diffusion coefficient of nitrogen in sea water is somewhat lower than that of oxygen in fresh water, and that the mathematical analysis of this problem by Col. Black and the writer, if not supported, is at least not invalidated, by Dr. Adeney's figures.

Discussions of theory and of laboratory investigations on such a complex problem as this are likely to be futile, unless the results can be controlled and checked against actual conditions. The problem of re-aeration is an enormously complicated one, but, fortunately, New York Harbor provides a full-scale experiment, and the various investigations which have been made on its waters have furnished a considerable mass of analytical and other data. It will be of interest, therefore, to inquire to what extent Dr. Adeney's values for re-aeration in a harbor water as a whole differ from those of Col. Black and the writer, and if there be serious difference, how the two sets of values compare with existing conditions. Dr. Adeney states* that any pollution which would not cause a greater rate of absorption than 0.055 c.c. of oxygen per liter per hour would exercise no apparent effect on the aeration of a tidal water. The context shows that he means the condition of aeration, not the rate of re-aeration. Col. Black and the writer have already discussed the errors of this point of view, which gives no consideration whatever to the initial conditions of aeration or of depth. It must be obvious that a saturated water will absorb no oxygen; that there must be a reduction from the normal condition of aeration before absorption begins; and that, furthermore, the greater this reduction the greater the absorption. The figure quoted refers to a mixture of salt and fresh waters in equal proportions, and the value for a mixture containing 75% of sea water is 25% greater. This is the statement taken literally and without question by Edlow W. Harrison, M. Am. Soc. C. E., upon which he based his computation cited by Col. Black and the writer (page 49) to show that the waters of New York Harbor could provide for the sewage of 60 000 000 people without reducing the normal oxygen. Dr. Adeney, in his New York report, makes no attempt to treat the question quantitatively, but points out the serious

* British Royal Commission on Sewage Disposal, Fifth Report, Appendix VI, p. 67.

variation in important factors, and believes that the rate of re-aeration may vary from zero to such a magnitude as to be of practical importance. His only approach to a quantitative statement is to the effect that rates of 0.067 to 0.077 c.c. of oxygen per liter per hour are probably maxima. The average of these two values is equivalent to 0.86 lb. per million gallons per hour, a figure which will be used later for comparison. This value refers to 6-ft. depths, and Dr. Adeney concludes that in greater depths the rate will vary inversely, so that in 12-ft. depths we may assign 0.43 lb. as Dr. Adeney's maximum. In Dr. Adeney's original report, 6-ft. columns were taken to represent actual conditions in practice. Recognition of the depth factor is apparently due to the Black and Phelps formula. Col. Black and the writer also realized the complex nature of the problem, but undertook a definite solution which would involve the most important variables. They estimated average rates of absorption, in 12-ft. depths, of about 0.045 lb. per million gallons per hour, although this figure varies considerably in the various portions of the bay.

Comparison of these figures with Dr. Adeney's maximum, after modification for depth, shows them to be about one-tenth as great. Dr. Adeney confesses his inability to determine an average rate, but claims that it is "of such magnitude as to be of great practical importance." Our figures are average rates, and represent an immaterial quantity in the treatment of the whole subject. While Mr. Harrison's estimate would probably be an unfair statement of Dr. Adeney's views, as now modified in his New York report, it should be noted that it is the only computation that has ever been made to show the value of the "streaming effect," and Dr. Adeney has made no attempt to correct it, or to replace it with a more conservative computation. The Black and Phelps formula has been applied to the conditions existing in New York Harbor, and, in conjunction with Col. Black's excellent mathematical analyses of the tidal conditions existing, has been used as a basis for a computation of the actual condition of the waters of various parts of the harbor. Instead of providing for 60 000 000 people without reduction of the normal oxygen of the waters, we find the effect of re-aeration to be practically negligible. If there be no reduction from the normal there will be no re-aeration. If 30% of the normal oxygen of the harbor waters be used up in sewage oxidation, so that the waters stand at 70% of saturation, the additional aeration

will amount to less than 1% of saturation in 12 hours. Under these conditions, oxygen obtained from the atmosphere by absorption would care for the sewage of about 250 000 people, out of a total of 7.4 millions which are provided for by the 30 per cent. The fact that, with less than this population contributing sewage to-day, the waters of the harbor do not stand at their normal oxygen content, but are on the average already reduced to approximately 70% of saturation, is of interest. The calculated condition, assuming practically no reaeration, agrees closely with the facts. A more severe test of our conclusions, however, is possible. The Metropolitan Sewerage Commission made 1 870 analyses for dissolved oxygen in and near New York Harbor during 1911. This work was confined mainly to typical cross-sections. Table 3 shows the original computations by Col. Black and the writer, in comparison with a summary of the Metropolitan Commission's results.

TABLE 3.—Dissolved Oxygen in New York Harbor Waters.

A. Calculated by Black and Phelps, February, 1911.

B. Analyses by Metropolitan Sewerage Commission for the Year 1911, Reported August 1st, 1912.

Location.	Percentage of saturation.	
	A	*B*
Lower Bay.		
End of flood	100	100
End of ebb	71	74
Upper Bay.		
End of flood	76	79
End of ebb	54	55–56
Hudson to 96th Street.		
End of flood	54	56
End of ebb	48	54
East River.		
Mouth to Hell Gate.		
End of flood	53	55
End of ebb	71	57
Hell Gate to Throgs Neck.		
End of flood	71	57
End of ebb	81	86

The computations refer to the areas indicated in Table 3, while the Commission's figures are necessarily for cross-sections. In each case the value for the area in question at the end of the flood is compared

with the average figures for flood and ebb at the cross-section down stream from the area in question; and at the end of the ebb with similar values for the cross-sections up stream from the area in question. In the single case of the Hudson River at 96th Street at the end of the ebb there are no up-stream data available. The figure given is the average value for the ebb flow at the mouth of the Hudson. Obviously, the character of the water remaining in this section at the end of the ebb is worse than the average ebb flow, a fact which is indicated in the discrepancy of these two figures. With a single exception, the remaining figures show a striking agreement. There is evidently something wrong with our value at the region of Hell Gate, which has influenced the computed value of the ebb-waters below Hell Gate and the flood-waters above. It might be suggested that there is an influence of the Harlem River shown here which was not properly taken into account in our computation.

The agreement between the computed and the actual conditions, although gratifying, is of itself of little practical moment. Its value lies in the fact that it demonstrates the substantial accuracy of the methods which have been used in the analysis of this problem, and makes it possible to use this same method in computing the probable effect of further increase in population, or of the discharge of additional quantities of sewage from regions not now tributary to New York Harbor. All the facts seem to support the view that aeration in New York Harbor is an immaterial factor in the whole problem of sewage disposal.

Upon the grounds of theory, experiment, and experience, therefore, the writer, much to his personal regret, finds himself obliged to differ from Dr. Adeney, whose personal acquaintance he has the honor to enjoy and for whose scientific work he has the most cordial regard.

DISCUSSION

W. E. Adeney, Esq. (by letter).—In this paper Professor Phelps does not offer any additional experimental evidence in support of the formula which Col. Black and he, in their Joint Report of 1911 to the Board of Estimate and Apportionment, have proposed for calculating the rate at which de-aerated waters become re-aerated, when exposed to natural conditions, in terms of time, depth, initial dissolved oxygen content, and the diffusion coefficient of dissolved oxygen in water. Dr. Adeney.

The fundamental conception underlying this formula is that the re-aeration of de-aerated waters is effected by the absorption of oxygen by the layer of water exposed to the air; that the subsequent transmission of the dissolved oxygen throughout the underlying masses of water takes place strictly in accordance with Fick's law of diffusion; and that, as far as appreciable effects are concerned, no other natural agency or factor need be considered.

The only experimental evidence, which the authors have advanced in arriving at their proposed formula, are some figures, which they have put forward as indicating the diffusion coefficients of dissolved oxygen in distilled water at certain stated temperatures. These figures have been determined by a method which the authors of the formula designed especially for the purpose, but the working details of which have only been slightly indicated in the report referred to.

The experimental difficulties in the way of determining accurately the coefficients of dissolved gases in water are very great, and Hüfner, so far as the writer knows, is the only observer, among the few who have ventured into this field of investigation, who has endeavored successfully to overcome the most serious of these difficulties.

The writer has already pointed out, in Section II of the Report which he had the honor of presenting to the Metropolitan Sewerage Commission of New York in 1911, that the method in question does not determine the diffusion coefficient, but the rate of re-aeration of the water experimented with by quite a different factor, that is to say, by the downward "streaming" of dissolved oxygen from the exposed surface of the water; and, inasmuch as the water used by Col. Black and Professor Phelps in their experiments was not distilled, but contained decided quantities of mineral matters in solution, this "streaming" process must have been very rapid, with the result that the values which they obtained for the diffusion coefficients in distilled water were very much too high.

A rough estimate of the relative rates at which columns of de-aerated sea water may become re-aerated, by (*a*) downward "streaming" from the surface exposed to the air, and (*b*) by true diffusion

Dr. Adeney. in accordance with Fick's law, may be obtained from the results of an experiment which the writer has described, as follows:*

"A U-shaped tube, each limb 1 600 mm. long and 50 mm. diameter, was filled with boiling sea water and immediately closed air-tight with indiarubber corks. When the water had cooled, the tube was further exhausted by means of a mercury pump. One limb was then uncorked, and the water in it saturated with atmospheric gases by a current of air drawn through the surface-layer to a depth of 50 mm. for 48 hours. After this, a sample was drawn from the bottom of the open limb, and one from the lower portion of the closed limb, about 200 mm. from the bottom. The gases were extracted from these and the nitrogen in each determined, with the following results [expressed in cc. at 0° C., and 760 mm. bar., per liter]:

Open Limb.	Closed Limb.
12.12	1.49

Six months later another sample was collected from the lower portion of the closed limb, as before, and the nitrogen in it determined; it was 1.73."

These results show clearly that, though the re-aeration of a column of de-aerated sea water, 1 600 mm. deep, and of uniform density, is relatively rapid, so that it may be completely saturated with air in 48 hours when evaporation freely occurs from the surface, it is so extremely slow when dependent on simple diffusion, that it may be altogether neglected in considering questions of the re-aeration of sewage-polluted tidal waters. It may be noted that the temperature of the laboratory during the first two days of the experiment described varied between 10° and 12° cent.

Professor Phelps himself has admitted that his method is faulty. On page 1629 he states in reply to the writer's criticisms:

"There are admittedly imperfections in the process, and the results are not of extreme scientific accuracy. For this reason, the work, which was originally carried out as a contribution to pure science, was not published as such."

By way of self-defense, he continues to remark that the accuracy of the method is sufficient "for an engineering discussion involving so many unknown factors, as does the New York Harbor problem."

Surely, the problem of determining accurately the diffusion coefficients of dissolved oxygen in water cannot be placed under the category of "involving so many unknown factors." The conditions necessary for the accurate determination of the diffusion coefficients in question are certainly quite well understood, although it may not be easy experimentally to ensure them; and, inasmuch as the whole superstructure of the formula depends on the accurate determination of these diffusion coefficients, the writer ventures to express the opinion that Professor Phelps would have been well advised had he submitted

* Section I in Appendix VI to the British Royal Commission's Fifth Report, p. 60.

a detailed description of his method to the consideration, in the ordinary way, of his brother chemists, before applying it to the solution of a problem of such wide and far-reaching importance as that of New York Harbor, affecting, as it does, the well-being of many millions of people, as well as the interests of a vast and rapidly growing commercial center. Had he done so, he no doubt would have had the benefit of the criticisms of other workers, and would not have been left long in doubt as to the nature of the "unknown factor" which has vitiated his coefficient determinations. The writer ventures to think that he has given the true explanation of this unknown factor, in his criticisms* of the method, in which he has shown that the experimental basis of that method is quite erroneous. Dr. Adeney.

Professor Phelps complains (page 1634) that the writer has not suggested a general expression applicable to the calculation of the average rate of re-aeration of tidal waters by "streaming." This was not done, because, to have attempted anything in the way of a working formula for the purpose, he would have had to follow the very undesirable course of making broad assumptions, for which there exists at present no scientific basis. In fact, he would have had to follow the course which he complains that Col. Black and Professor Phelps have adopted in arriving at their proposed formula. The writer is quite willing to admit that such a course would be warranted for the purpose of raising a discussion on the question, if restricted to purely scientific interest, but certainly not in connection with one of such vast practical importance as that of the powers possessed by the tidal waters of New York Harbor for the disposal of sewage matters, within limits of safety to public health, and to the amenities of their surroundings.

The writer has preferred to follow the usual custom by publishing a description of his experiments on "streaming," although they were simply pioneering experiments, and has to acknowledge that he has received the benefit of some helpful criticism from Professor Phelps. He gladly takes this opportunity of expressing his appreciation and thanks for that criticism. The writer has also to express his indebtedness to Professor Phelps for pointing out that the results of his experiments do not bear the precise and general value that has been placed on them by some writers in the United States, because, as already stated, they are only of a pioneering character, and are necessarily only true for the conditions of the particular experiments described.

The fact, however, that the proposed formula rests on an erroneous experimental basis, is not the only serious objection that may be urged against the position that has been taken by Col. Black and Professor Phelps on this question of re-aeration.

*Report to the Metropolitan Sewerage Commission of New York, 1911, p. 83.

Dr. Adeney

With reference to the particular question of the re-aeration of tidal waters, Professor Phelps assumes that the same factors obtain with sea water as with fresh water, and that it is effected solely by diffusion, according to Fick's law, from the exposed surface downward.

According to Professor Phelps' strongly expressed opinion, however, the surface layers of inshore tidal waters are fresher by reason of admixture with land waters, and, therefore, less dense than the heavier salt water below. In other words, the layers of water concerned in actually dissolving oxygen from the air, for subsequent diffusion below, are less dense than the lower layer of the water. Notwithstanding this, Professor Phelps assumes that diffusion will proceed according to Fick's law, just as it would were the water of uniform density throughout its depth. Again, he gives no experimental data in proof of this, and the writer knows of none himself. In the absence of direct evidence to prove it, Professor Phelps is not justified, in the writer's opinion, in stating that, because Fick's law holds good for the diffusion of dissolved oxygen in a liquid of uniform density, it must necessarily do so in one increasing in density in the direction of diffusion. Under these circumstances, the writer is entitled to hold the contrary view.

One further assumption, which Col. Black and Professor Phelps make, demands notice. It is stated on page 56 of their original report, and is as follows:

"Since the fundamental law of aeration is one of diffusion, and, furthermore, since the surface of any body of water is practically saturated at all times, it must be evident that no mere surface action can by itself affect the rate of diffusion."

No experimental evidence of any kind is given in support of this statement. The question is one on which the writer has made a considerable number of experimental observations since he published the description of his pioneering experiments on "streaming," all of which prove beyond doubt that the surface of a body of water, when exposed to atmospheric conditions, does not become saturated at any time as long as the layers immediately below it remain only partly aerated.

The writer has definitely found, by careful experimental observations, that, as oxygen is dissolved by the surface of any quiescent body of water, when de-aerated or only partly aerated, it passes practically as quickly downward through the lower layers, so that the surface does not become saturated until the dissolved oxygen content of the layers below it approach the condition of saturation.

He has found this to be true both of fresh water and of sea water. The only marked difference between the two is that in the sea water the transmission downward is quicker than in fresh water, when the surfaces of the two bodies of water are exposed to the air so that evapo-

ration can take place freely from them; and *vice versa* when the conditions are such as to prevent evaporation from the exposed surfaces. Dr. Adeney.

The writer has also found that when the surfaces of de-aerated columns of fresh water are exposed to the air, so that evaporation can take place freely from them, and when the columns themselves are maintained at a constant and uniform temperature, the downward distribution of the dissolved oxygen is so nearly uniform, for depths of about 1 000 mm., that the oxygen content of the water at that depth has been found to be nearly equal to that of the surface layer at various stages during the process of re-aeration.

Professor Phelps admits the correctness of the writer's observations on "streaming" in his experiments with columns of sea water, but assumes that "streaming" does not affect the practical question of the re-aeration of tidal waters, because it was caused, as he explains, in the case of the writer's experiments, by the formation of concentrated layers of water, the result of evaporation from the exposed surfaces of the experimental columns; and he assumes that the "streaming" would not occur in tidal waters, when the surface layers are fresher and, consequently, denser than the lower layers of sea water.

Again, Professor Phelps gives no experimental evidence in support of this assumption, and, in the absence of definite evidence, he is no more entitled to make it, than he is to assume that diffusion will occur according to Fick's law under the same conditions.

The writer has already pointed out that if a marked stratification obtains in a tidal water, and the land water lies on the surface of the sea water, the sewage matters, which are themselves carried by fresh water, will gather to the surface layer also and will there be re-aerated chiefly by "streaming."

The New York Harbor waters, however, are not thus stratified. The Metropolitan Commission states:*

"The general configuration of the bottom and of the main channels of the harbor is such as materially to aid in keeping the water well mixed. * * * The water is usually more saline at the bottom than at the top, but the difference is usually slight and there is apparently no sharp line of demarcation."

Professor Phelps would be justified, no doubt, in assuming that "streaming" would not extend to the lower layers of a tidal water, the surface layer of which consisted practically of wholly fresh water, and the lower layers of undiluted sea water, but he is certainly not justified in assuming, without experimental observation, that it would also not occur in tidal waters in which, as in New York Harbor, the water at a depth of 20 ft. contained, on an average, only 6% more sea water than at the surface. The experiments which the writer has made certainly do not justify such an assumption.

*1912 Report, pp. 26 and 27.

Dr. Adeney. The writer does not think that any useful object can be attained by following Professor Phelps in his detailed discussion as regards so-called calculated and observed results, because, until the scientific basis on which such a discussion can be usefully carried on has been definitely settled, it can only be juggling with figures to attempt to do so.

In his Report to the Commission (page 81), the writer has mentioned the chief factors, as far as they are known, which are concerned in the re-aeration of tidal waters exposed to natural conditions, and gives reasons for believing that:

"The rate of re-aeration must therefore be an extremely variable quantity from day to day, and even from hour to hour."

Again, on page 87, he states:

"It is quite evident that under their combined influence [*i. e.*, the factors concerned in re-aeration] the rate [of re-aeration] may vary practically from zero to a value of such magnitude as to be of great practical importance especially during summer season.

"It need scarcely be pointed out, however, that a good deal of work has yet to be carried out, both in the laboratory and on the open water, before a definite value can be put to the extreme rate at which re-aeration under the most favorable conditions would occur, or even to the average rate—both would certainly vary with different classes of inshore waters."

Mr. Phelps. EARLE B. PHELPS, ASSOC. AM. SOC. C. E. (by letter).—The writer wishes to thank Dr. Adeney for his frank criticism, which makes it possible to define clearly the points at issue. There is apparently complete agreement on the two propositions that diffusion alone is of no material advantage in the re-aeration of polluted tidal waters and that the phenomenon of "streaming," if it takes place, effects re-aeration to a material extent. This being the case, further discussion of the accuracy of the determination of the diffusion coefficient becomes irrelevant. The writer sees no reason to alter his opinion that figures of this degree of accuracy, although admittedly too imperfect for use in a journal of physical chemistry, still have a legitimate use, especially when the probable magnitude of the error is stated. A review of the original data, in the light of the criticism made, leaves the writer still of the opinion that Fick's law holds in this case, that the mathematical analysis of the problem is sound, and that the value of the coefficient as determined has a degree of accuracy better than that obtaining in other important engineering values that enter the problem, such as the measurement of stream flow and of sewage.

There remains, however, as relevant to this discussion, the single question: "Does streaming take place in New York Harbor?" Dr. Adeney apparently accepts the writer's explanation of the nature of the streaming phenomenon, so far as saline waters are concerned, in

Mr. Phelps.

that it is a gravitational movement of the more dense layers produced at the surface by evaporation. He objects, however, to the "assumption" that this process is not taking place in a body of water in which the density actually increases from the surface downward. Surely this is more than an assumption, and it calls for no experimental proof that lighter layers of water will not sink through heavier layers by the action of gravity. The quotation from the Metropolitan Sewerage Commission's report is not a sufficient basis for the statement that the waters are not stratified, if by stratified we understand an increase of density downward. The figures and diagrams of that report show such a condition almost universally, and the writer's experience, in the Charles River Basin at Boston, and elsewhere, indicates such a condition to be normal in estuary waters. Whether the density gradient be great or small, and whether or not there be a distinct line of demarcation (which is often found), the mere existence of a density gradient downward is obviously incompatible with a downward gravitational streaming of water.

Dr. Adeney does an unintentional injustice to our formula when he states at the outset that it determines the rate of aeration in terms of time, depth, initial dissolved oxygen content, and diffusion coefficient; and (later) that "no other natural agency or factor need be considered." A study of the method of using the formula will show that the time factor is the mean time between uniform mixings, and that for its determination one must know all the physical conditions and natural agencies that assist in mixing the water, such as wind, tidal mixing, and the effect of passing boats. This work is now being extended by the determination of the time factor for streams of various types. Knowledge of this factor will make it possible to determine the rate of re-aeration of such streams. The formula has already been made use of in this way by the experts of the Royal Sewerage Commission of Great Britain, in its Eighth Report. The results show at once that the possibilities of re-aeration in such streams are far greater than in the case of deep estuaries.

The writer cannot agree that the correspondence between calculated and observed results in New York Harbor is only a "juggling with figures." The calculated results are based on the practical negligibility of re-aeration. The fact that they agree to a remarkable degree with the results of an extensive series of analyses made during the following year is certainly the most desirable kind of "additional experimental evidence," the absence of which Dr. Adeney regrets so keenly. If these calculations had been made by omitting all reference to re-aeration, the results would not have been altered materially. They do not involve any of the disputed points, and, without further reference to the other matters, they alone show the practical non-importance of re-aeration in New York Harbor.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1273

THE USE OF CEMENT FOR EXCLUDING WATER FROM OIL SANDS IN DRILLING WELLS.

By Paul M. Paine, Assoc. M. Am. Soc. C. E.

In connection with the drilling of oil wells, cement is being used in increasing quantities for excluding from the oil sand the water in strata nearer the surface. This water, if not prevented from going down into the oil sand, will displace the oil, because of its greater specific gravity, and eventually ruin the well. Unfortunately, the damage does not end here, for experiences in the older fields have shown that this flooding of the oil sand spreads gradually over wide areas, depending mainly on the porosity of the sand and the gravity of the oil, and, unless checked in some manner, will ruin neighboring wells. For this reason the encroachment of water into the oil-bearing strata is a serious problem in any oil-field, and is particularly difficult to overcome where the water strata are directly above or below the oil-bearing sand.

The mechanical difficulties and expense connected with this problem have led careless operators to take long chances at times, and, in many of these cases, where the work has not been successful, there has been serious injury, not only to such wells, but also to adjoining properties. A measure of State relief has been obtained in some States where laws have been passed requiring operators to remedy such defects, but it is generally agreed that the only permanently

satisfactory method is to shut off the water effectually at the time the well is being drilled and before the hole has been carried into the oil sand.

The development of deep drilling, which has come with the introduction of the use of casing, has seen marked advances in methods, and it is not unusual to finish wells with a 6-in. hole at a depth of 4 000 ft. or more. In such wells several "strings" of pipe may be used, the first and largest being carried down from the surface until it is "landed" at a suitable depth, or until it is "frozen" by caving material falling in against it from the walls of the hole and binding it so that it cannot be moved farther, either up or down. The next "string," the collars of which will run inside of the first one, is then put in, and drilling is continued until this second string is either landed or frozen, and so wells frequently contain strings of 15½-in., 70-lb., 12½-in., 40-lb., 10-in., 40-lb., 8-in., 28-lb., 6¼-in., 20 or 26-lb., and 4½-in., 12½-lb., lap-welded, screw casing. At the bottom of each of these strings there is a shoe with an outside diameter slightly greater than that of the pipe collars, so that, as the pipe is lowered, the collars will be sure to pass through any opening through which the shoe has passed.

Preventing the water from reaching the oil is effected by landing a tight string of pipe, called the "water string," at a level known to be below the lowest water stratum and above the oil sand. This is done before the hole is drilled into the latter, and, wherever possible, the water string is landed on a "shell," the general oil-field term for any hard compact rock. This is illustrated in Fig. 1. In some fields all that is necessary is to stop drilling and drive the pipe into this shell; the well is then baled dry, and, after standing a day or two, a baler is run in to ascertain whether or not the pipe is holding the water back. If water has come in, it means that the bond between the casing shoe and the shell is not tight, or that the casing leaks. The cause of the leakage may be determined by special devices, and then a suitable remedy may be applied. Often, however, a hard shell in which to land the casing cannot be found, and the pipe is stopped when the shoe is in some soft measure such as shale. In either case, it frequently happens that wells which have shown a satisfactory water test at the time of drilling, have developed water later, due to

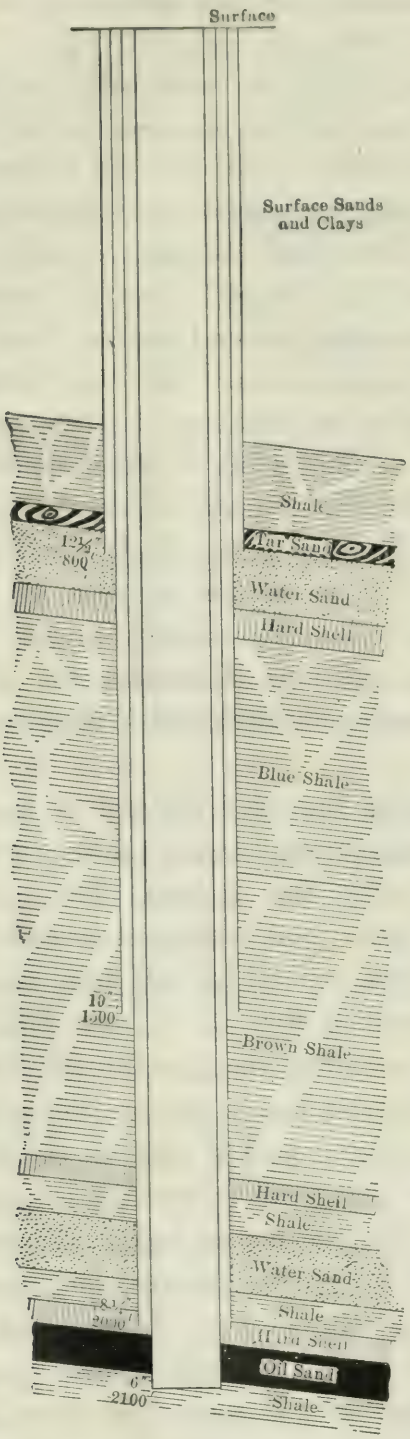

Surface
Surface Sands and Clays
Shale
Tar Sand
Water Sand
Hard Shell
Blue Shale
Brown Shale
Hard Shell
Shale
Water Sand
Shale
Hard Shell
Oil Sand
Shale
12¼"
800
10"
1500
6"
2100

FIG. 1.

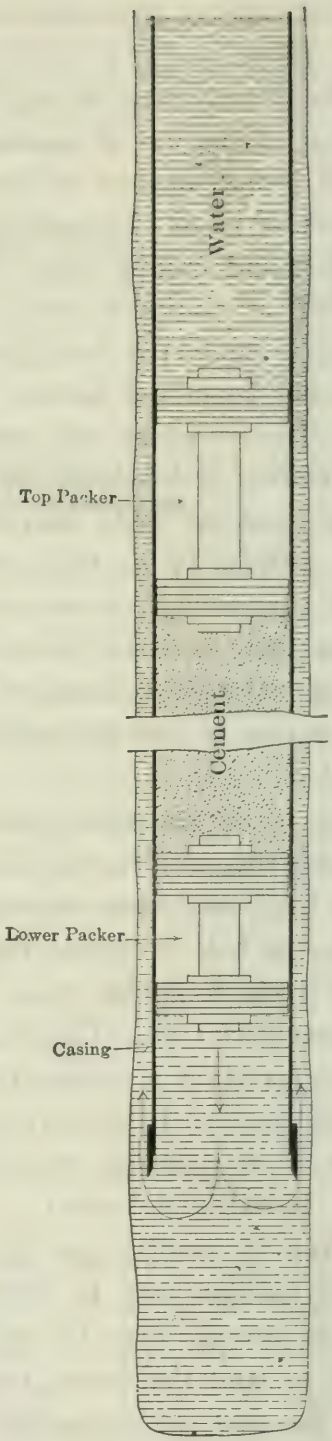

Water
Top Packer
Cement
Lower Packer
Casing

FIG. 2.

gradual leaching through the interstices of surrounding porous measures. This uncertainty led to the search for a more thorough method of sealing off the water, and the use of cement has come to be recognized as the only positive remedy. The problem, then, is one of placing from 2 to 8 tons of cement between the casing and the wall, at the bottom of holes ranging in depth from 600 to 4 000 ft.

In all the methods described herein, when the hole has reached the desired depth at which to cement, it is reamed out wider for 60 or 70 ft. above the bottom with an under-reamer, a tool which widens the hole with cutting lugs that spring out when lowered below the casing shoe. This gives additional room for the cement and insures a better bond. The hole is also cleaned of mud and cuttings, and fresh water is run in. The simplest method of placing the cement is that known as "baling" it in. The hole is filled with water to the top of the casing, and the latter is raised until the shoe is 10 or 12 ft. from the bottom. The cement, mixed to a thick grout, is then run in in a specially devised baler which dumps when it reaches the bottom. When from 1 to 2 tons (dry weight) have been placed in this way, a plug is screwed into the casing collar at the top, and the casing is lowered. As the casing is full of water, the weight of the pipe forces a large portion of the cement out into the formation and up between the casing and the wall of the hole, as the casing shoe is lowered to the bottom. As a final precaution, the casing is driven, in order to force the shoe into the bottom as far as possible. Instead of using a baler, some operators prefer to run in the cement in a series of long narrow bags tied to the bottom of the drilling tools; when the bottom is reached, a few strokes of the tools loosen the bags and break them up so that the cement is freed. In connection with these methods the patented Baker "cement plug" is sometimes used instead of the one screwed into the top of the casing before it is lowered. This plug is of light cast iron, made so that it may be hung from the baler with soft rope and run down inside the casing. When lowered below the casing shoe and then raised with a slight tension, a set of clips catch on the shoe, and the bottom opening of the casing is effectually closed. The casing is then lowered, the cement is forced up on the outside, and the baler is loosened by a stronger pull which breaks the soft rope. The plug, being of cast iron, is easily drilled up.

A method of particular value in very deep wells, or in those in which the water string tends to "freeze" unless moved at frequent intervals, is that known as the disk method, in which patented packers are used. This requires that a "circulation" be obtained, that is, that there be a free passage for fluid pumped down inside the pipe to come to the surface on the outside. In order to obtain this, a 10 by 5 by 12-in. pump of the type used in the oil-fields for pumping mud in drilling wells with the rotary system, is set up and connected to the casing head with a section of 2½-in. pressure armored hose, as shown in Fig. 3. It is frequently necessary to work up to a pressure of several hundred

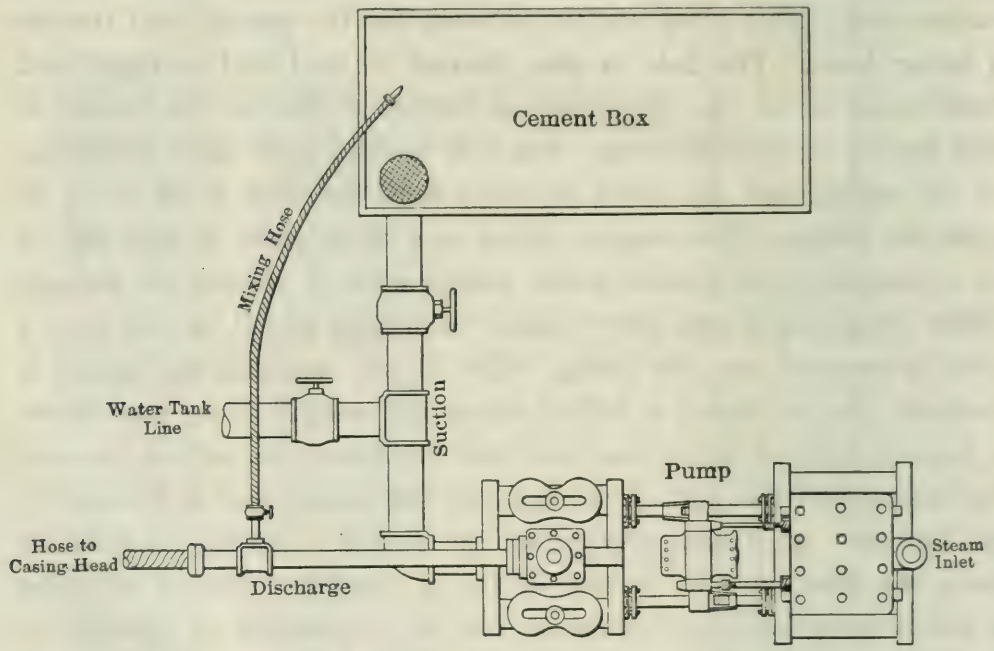


FIG. 3.

pounds before the fluid will create a passage on the outside of the pipe from the casing shoe to the surface. When this circulation has been obtained, the casing is suspended so that the shoe is 4 or 5 ft. from the bottom and enough fresh water is pumped in to insure a thorough cleaning of the bottom. The packers are of wood or of cast iron, and have heavy canvas or rubber washers which fill the space inside the casing, as shown in Fig. 2. The lower one is about 3 ft. long, and, after it has been placed in the casing, the pump suction is connected with the box in which the cement has been mixed just thin enough to pump readily. The cement is then pumped in against the lower packer, forcing it down until the cement box is drained; the top

packer follows, and water is then pumped in against the latter, pushing the combination of lower packer, cement, and top packer down the pipe. When the lower packer passes the casing shoe, it falls to the bottom of the hole and permits the cement to pass around the shoe and up on the outside of the casing. The top packer is of such length that when it has forced the cement out of the pipe and it strikes the lower packer, its top set of washers will not have passed out of the casing, and, as it cannot move farther, it will raise the pressure inside the casing and stop the pump. The casing is then landed on the bottom, the cement has been placed outside of the pipe at the bottom of the hole, and the packers, like the cement plug, are easily drilled up after the cement has set. The main objections to this method are the danger of the packers sticking while going down inside the casing and the fact that the lower packer may fall to the bottom in such way as to prevent the casing shoe from being landed squarely on the bottom, and thus get under it so that the bond is broken when the packer is drilled up.

The following method is in more general use, as it has seemed to secure better results and has no patented features: In this, as in the packer method, the principle of pumping in the cement under conditions that force it up on the outside of the casing is used, but this is done with an extra string of small pipe, usually 2½-in. tubing, which is lowered inside of the casing. After the hole has been drilled to the desired depth, reamed, and a circulation obtained, the casing is suspended with its shoe about 6 or 8 ft. from the bottom; the hole is filled with water to the top of the casing, and a string of tubing is run in until its bottom is slightly above the casing shoe. The packing head, shown in Fig. 4, is screwed into the casing collar at the top, and the gland in this head acts as a stuffing-box around the tubing. Then, with the casing full of water, when water is pumped in through the tubing, its only course is up on the outside of the casing. The surface arrangement for the pump is similar to that for the packer method (Fig. 3). On the bottom of the tubing has been placed either a bushing or a swedge nipple, reducing the opening from 2½ to 2 in. First, fresh water is pumped into the tubing, and this is followed with the cement grout; as soon as the cement is all in, the plug in the tubing tee above the packing head is unscrewed, and a wooden plug with a washer on it is dropped in on the cement. The tubing plug is then

replaced, and water is pumped in against the wooden plug, chasing the column of cement down the tubing. The water in the casing is incompressible, and the cement must travel around the shoe and up on the outside. The purpose of the swedge nipple at the bottom of the tubing is to show when the cement has passed out. The plug is stopped, and the pressure on the gauge at the pump goes up. The casing is then lowered to the bottom, and the tubing is pulled out as rapidly as possible so that whatever cement has remained inside the casing may be baled out before it has had time to set. The cement is then allowed from 10 to 20 days in which to set, after which the well is drilled a few feet ahead of the shoe and then baled dry. If no water has come in at the end of a 24-hour period, it shows that the work has been successful, and drilling operations are resumed.

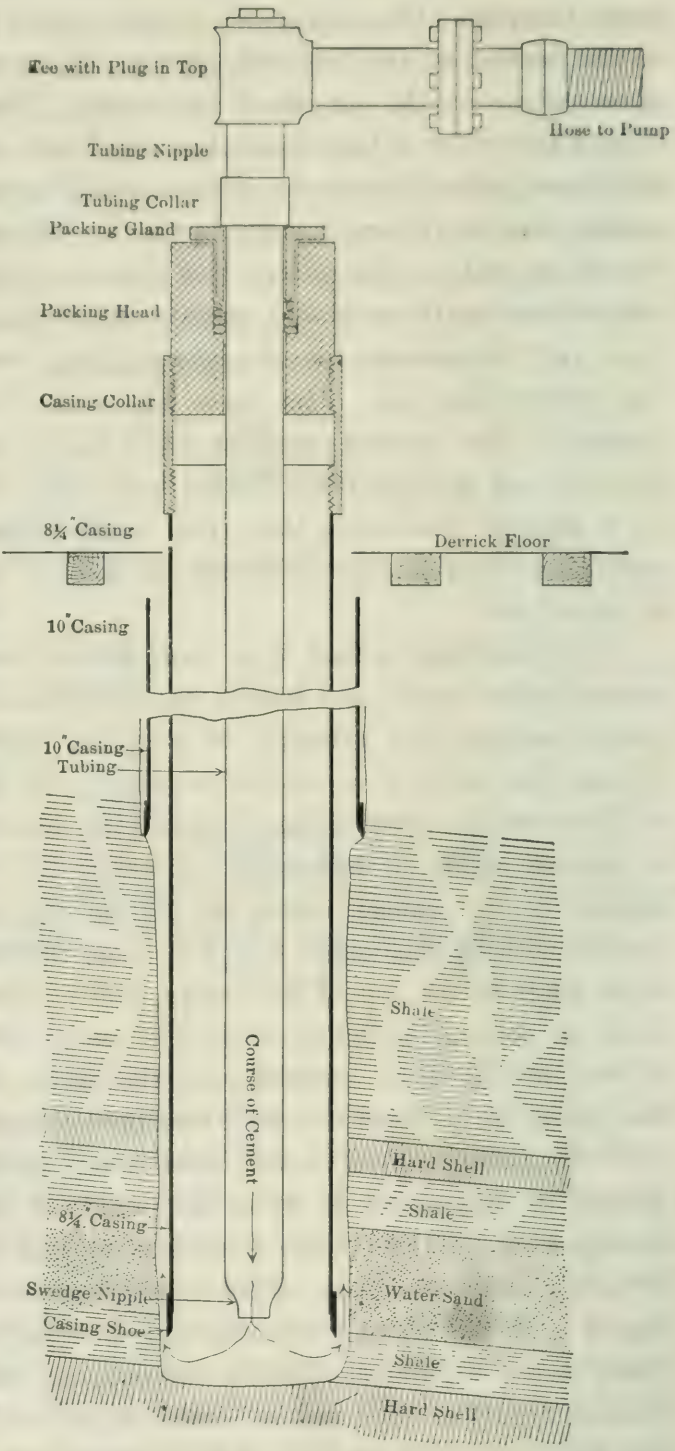


FIG. 4.

In one well cemented by the writer, a still lower water sand was encountered. Back of a string of 8-in., 28-lb. casing, 2½ tons of cement had been placed, and as it was desired to shut off all the water with the 8-in. string and use a 6-in. string to finish the well, the former was cut off about 200 ft. from the bottom and the hole was re-drilled off to the side of the 8-in. pipe left in the old hole. This is known as "side-tracking," and the new hole goes off to the side of the old pipe and down directly beside it. This gave an opportunity to learn how far up, on the outside of the casing, the cement had set, and it was found to be 65 ft.

In the preliminary operation of securing a circulation, the fluid may come to the surface on the outside of the pipe, even though it is not coming around the shoe, if a leak exists in the casing. This may be determined by continuing to pump and lowering the casing to the bottom of the hole; if there is a leak, the circulation will continue; if the circulation has been entirely around the shoe, then when the latter is placed on the bottom, the fluid will be held in and the pump pressure will be raised.

From 2 to 6 tons (dry weight) of cement are generally used, depending on conditions; and even larger quantities are inserted where an unusual cavity is to be filled. Before being mixed with water the cement should be screened, and particular care should be taken that no pieces of bags or strings get into the suction line. Preferences for different brands of cement are found in different localities, but there appears to be little advantage in any one kind of cement, if the time of initial set is long enough to cover the period of mixing and pumping it, and of lowering the casing. Sand should not be mixed with the cement, and the grout should be given every opportunity to set; for this reason the hole should be made as free from mud, oil, and gas as possible, the latter especially tending to prevent the set by keeping the cement in motion.

It sometimes happens, particularly in new territory, that after the oil sand has been penetrated and the well is being drilled still deeper, another water sand is encountered, known as "bottom" water. Under such conditions, in the presence of active quantities of oil and gas, the work becomes more difficult. The most effective method is to pull up the casing until the shoe is a short distance below the bottom of the oil sand, and then build a "bridge" in the hole below the shoe by

running in broken pieces of brick and common manila rope. This bridge closes the hole and forms a landing place for the casing shoe after the cement has been run in. More care in gauging the quantity of cement to be placed is necessary in this case, in order that none of it may reach the oil sand and clog it, thereby tending to interfere with the oil production.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1274

ENGINEERING ACHIEVEMENTS AND ACTIVITIES OF NEW YORK CITY.*

By Alfred D. Flinn, M. Am. Soc. C. E.

Historical and General.

New York was founded as New Amsterdam, by the Dutch, in 1625, when two ships from Holland brought over settlers with their household and farming equipment and domestic animals. Under the direction of an engineer, Kryn Frederick, the town was begun by laying out a fort. There had been a trading post here, however, since 1613. In 1626, Peter Minuet bought the island from the Manhattan Indians for 60 guilders, equivalent to $24. By conquest, the settlement became an English colony in 1664.

Although an important port, it remained in a secondary place throughout the colonial period, and did not become the first city of the land, as to commerce and population, until 1770. The opening of the Erie Canal, in 1825, gave the City its first great impetus. Occupying originally only a small area at the southern extremity of Manhattan Island, the City, by growth and annexation, has greatly increased its territory and water-front. New York is now the greatest of all seaports, due to its geographical advantages, namely, its wonderful harbor and its easy lines for water and rail access to the interior of this great continent, to New England, and to the South Atlantic States. It is the terminus of the naturally cheapest route from the Mississippi Valley to the sea. It has a fine climate and a low death rate, 14 per 1 000 for 1912. It is one of the healthiest of the great cities of the world.

* An Illustrated Lecture, at the Reception given by the American Society of Civil Engineers, at the Society House, to the Foreign Delegates Attending the Sixth Congress of the International Association for Testing Materials, New York, September 5th, 1912. Enlarged for publication, and Revised, May, 1913.

Harbor and Traffic.

As shown by the map, Fig. 1, the harbor is naturally divided into several parts, well interconnected. At the entrance from the Atlantic is the Lower Bay, from which vessels pass through The Narrows into the Upper Bay, out of which branch the Hudson, or North, and the East Rivers, with the Harlem River linking them north of Manhattan Island. From the East River, through Hell Gate, commerce has access to Long Island Sound and New England waters, and the Hudson is navigable to Troy. On this map are indicated also the approximate locations of the principal artificially improved channels.

Into New York Harbor come the largest steamships afloat. In 1911 4 500 vessels, with a tonnage of 14 000 000, cleared for foreign ports. The total traffic of the harbor amounts to 100 000 000 tons annually. Vessels are sent to the port by many great oceanic, coastwise, and river lines. From 800 000 to 1 000 000 immigrants have passed through the Ellis Island station during each of the recent years. The imports of New York have amounted to $936 000 000, and the exports to $865-000 000, annually. Great numbers of tramp vessels come into the harbor from all over the world. Few more suitable anchorage grounds for naval vessels, where the public can see them conveniently, can be found than the Hudson River from 23d Street northward. Here the greatest fleet ever assembled has ridden safely at anchor. The river has a channel depth of 35 ft. or more as far as the City's northerly boundary. Harbor depths are indicated on Fig. 5.

Geology.

In The Bronx and on Manhattan Island above 14th Street, in general, mica schist, gneiss, or limestone, outcrops or is not far below the surface; but south of 14th Street the bed-rock descends steadily, and is everywhere covered heavily with glacial drift. Long Island and Staten Island are morainal, with rock usually at considerable depths, except for a ridge of poor serpentine outcropping on Staten Island, and one outcrop in the northerly part of western Long Island. Railroad, subway, and water tunnels, bridge and building foundations, and other engineering constructions are profoundly affected by this geological formation. Most picturesque of all the local geological formations is the great trap rock sill, known as The Palisades, extending from the harbor northward for 20 miles along

NEW JER
HUDSON R
UPPER BAY
BLACK TOM ISLAND
BEDLOE'S ISLAND
ISLS ISLAND
GOVERNORS ISLAND
BUTTERMILK CHANNEL
EAST RIVER
BROADWAY
THIRD
BLACKWELLS ISLAND
CENTRAL PARK
LINE A
LINE B-F-G-H OR I
LINE C
LINE D
LINE E
LINE J
LINE K
LINE L
LINE M
LINE N
LINE O
LINE P
LINE Q
LINE R
LINE S
LINE T
LINE U
LINE V
LINE W
LINE X
LINE Y
LINE Z
HELL GATE
FIFTH
SIXTH
SEVENTH
EIGHTH
NINTH
TENTH
ELEVENTH
TWELFTH
THIRTEENTH
FOURTEENTH
FIFTEENTH
SIXTEENTH
SEVENTEENTH
EIGHTEENTH
NINETEENTH
TWENTIETH
TWENTY-FIRST
TWENTY-SECOND
TWENTY-THIRD
TWENTY-FOURTH
TWENTY-FIFTH
TWENTY-SIXTH
TWENTY-SEVENTH
TWENTY-EIGHTH
TWENTY-NINTH
THIRTIETH
THIRTY-FIRST
THIRTY-SECOND
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THIRTY-NINTH
FORTIETH
FORTY-FIRST
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NINETY-FIFTH
NINETY-SIXTH
NINETY-SEVENTH
NINETY-EIGHTH
NINETY-NINTH
ONE HUNDREDTH
ONE HUNDRED AND FIRST
ONE HUNDRED AND SECOND
ONE HUNDRED AND THIRD
ONE HUNDRED AND FOURTH
ONE HUNDRED AND FIFTH
ONE HUNDRED AND SIXTH
ONE HUNDRED AND SEVENTH
ONE HUNDRED AND EIGHTH
ONE HUNDRED AND NINTH
ONE HUNDRED AND TENTH
ONE HUNDRED AND ELEVENTH
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ONE HUNDRED AND THIRTEENTH
ONE HUNDRED AND FOURTEENTH
ONE HUNDRED AND FIFTEENTH
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ONE HUNDRED AND TWENTIETH
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ONE HUNDRED AND TWENTY-SECOND
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ONE HUNDRED AND TWENTY-EIGHTH
ONE HUNDRED AND TWENTY-NINTH
ONE HUNDRED AND THIRTIETH
ONE HUNDRED AND THIRTY-FIRST
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ONE HUNDRED AND THIRTY-FOURTH
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ONE HUNDRED AND FIFTY-FIRST
ONE HUNDRED AND FIFTY-SECOND
ONE HUNDRED AND FIFTY-THIRD
ONE HUNDRED AND FIFTY-FOURTH
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ONE HUNDRED AND NINETIETH
ONE HUNDRED AND NINETY-FIRST
ONE HUNDRED AND NINETY-SECOND
ONE HUNDRED AND NINETY-THIRD
ONE HUNDRED AND NINETY-FOURTH
ONE HUNDRED AND NINETY-FIFTH
ONE HUNDRED AND NINETY-SIXTH
ONE HUNDRED AND NINETY-SEVENTH
ONE HUNDRED AND NINETY-EIGHTH
ONE HUNDRED AND NINETY-NINTH
TWO HUNDREDTH

PLATE LVII.
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1259A

MAP SHOWING
GEOLOGIC FORMATIONS
ALONG THE
PROPOSED LINES FOR CITY WATER TUNNEL

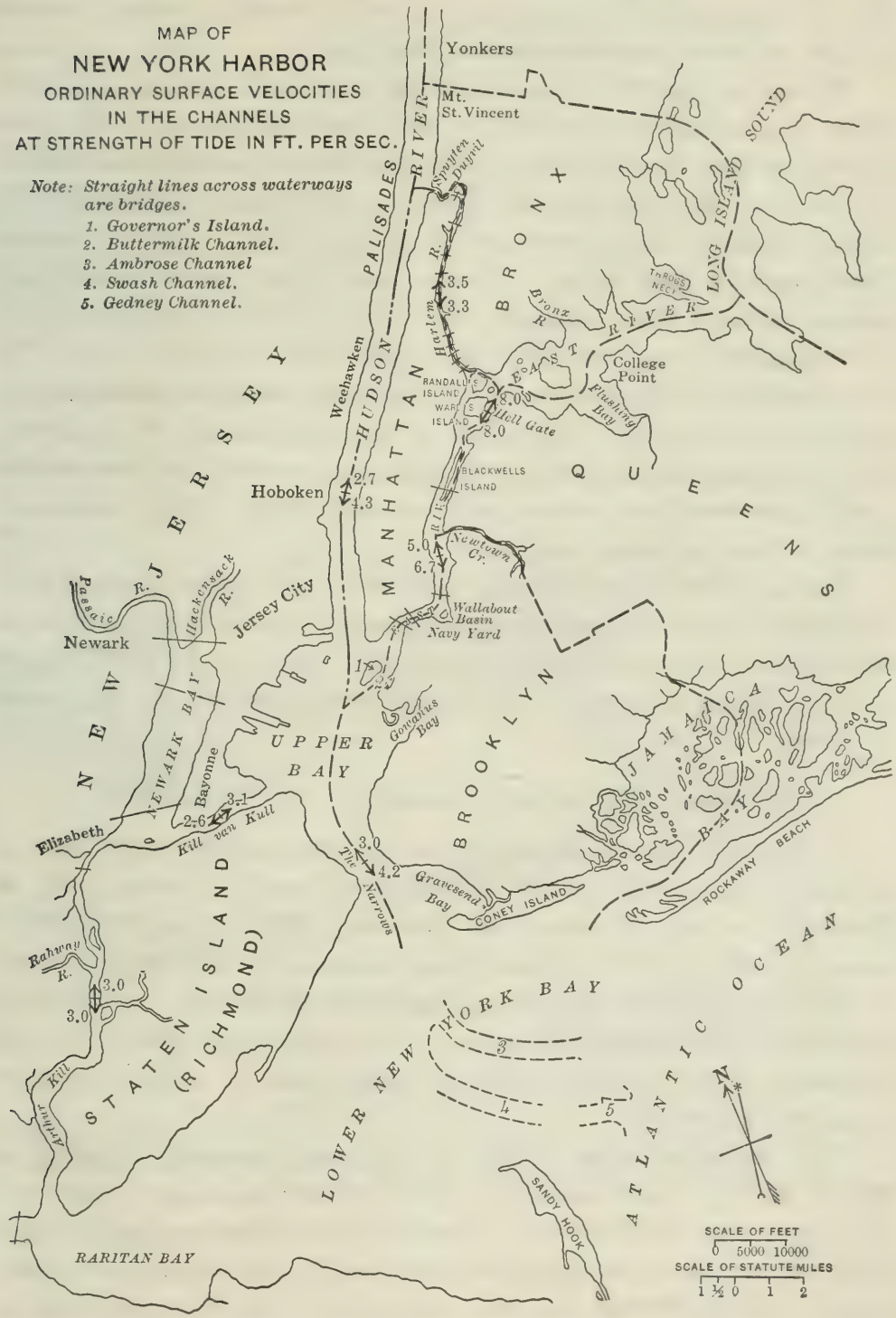
MAP OF
NEW YORK HARBOR
ORDINARY SURFACE VELOCITIES
IN THE CHANNELS
AT STRENGTH OF TIDE IN FT. PER SEC.
Note: Straight lines across waterways are bridges.
1. Governor's Island.
2. Buttermilk Channel.
3. Ambrose Channel
4. Swash Channel.
5. Gedney Channel.
Yonkers
Mt. St. Vincent
Sunnyside Drydock
Harlem R.
3.5
3.3
BRONX
Bronx R.
LONG ISLAND SOUND
THROGS NECK
RIVER
College Point
Flushing Bay
RANDALLS ISLAND
WARDS ISLAND
8.0
Hell Gate
8.0
EAST
BLACKWELLS ISLAND
QUEENS
Weehawken
HUDSON
PALISADES
MANHATTAN
NEW JERSEY
Hoboken
2.7
4.3
5.0
6.7
Newtown Cr.
Wallabout Basin Navy Yard
Passaic R.
Hackensack R.
Jersey City
Newark
NEWARK BAY
1.7
2.2
Gowanus Bay
UPPER BAY
BROOKLYN
JAMAICA B'AY
Bayonne
Elizabeth
2.6
3.7
Kill van Kull
3.0
4.2
The Narrows
Gravesend Bay
CONEY ISLAND
ROCKAWAY BEACH
ATLANTIC OCEAN
Rahway R.
3.0
3.0
STATEN ISLAND (RICHMOND)
Arthur Kill
LOWER NEW YORK BAY
3
4
5
SANDY HOOK
RARITAN BAY
N
SCALE OF FEET
0 5000 10000
SCALE OF STATUTE MILES
1 ½ 0 1 2

FIG. 1.

the west bank of the Hudson. Attaining a height of some 400 ft., this sill forms a barrier which has had to be pierced by many a tunnel for the railroads approaching from the West.

Within the past 20 years, explorations with wash, diamond, and shot drills, and subsequent excavations for large engineering works have greatly enriched our knowledge of the details of the geological structure. Tunnels and pipe lines for the Catskill Aqueduct and the rapid transit subways have developed north and south lines from Yonkers to Staten Island, and the great railway and gas tunnels, together with water and subway tunnels, have supplied data for east and west sections of the formations from the Hudson to the Suffolk County line. The rock contours for Manhattan Island have been well established and are shown on a few maps. As in most of these explorations the engineers have been assisted by expert geologists, the results have been intelligently interpreted and permanently recorded. Plate LVIII reproduces some of these geological sections.

During recent years extended observations for sewage disposal purposes have greatly enlarged the knowledge of tidal currents and kindred phenomena in the harbor, bays, and rivers. Figs. 1 and 5 give some of the most recent information concerning these currents, and the depths of water.

Resources, Industries, Geography, and Population.

To-day, the metropolis of the western hemisphere ranks first in population among the cities of the world, and is second only to London as a financial center. In February, 1912, the capital and undivided profits of its banks and trust companies totaled $581 000 000; at the same date, the deposits aggregated $3 226 000 000. In addition, the deposits in savings banks in the greater city exceeded $1 000 000 000. Summing these, it appears that the grand total of deposits approximates $4 500 000 000.

The estimated total value of real and personal property is $10 000 000 000; total bonded debt, $800 000 000; gross public indebtedness, more than $1 000 000 000. Against this indebtedness can be offset the 5 500 parcels of real estate owned by the City, with an estimated valuation of $1 300 000 000, and the equipment of the City departments, on which a value of $40 000 000 has been set. Central Park is valued at $380 000 000, and Bowling Green, for which the annual rental was,

PLATE LVIII.
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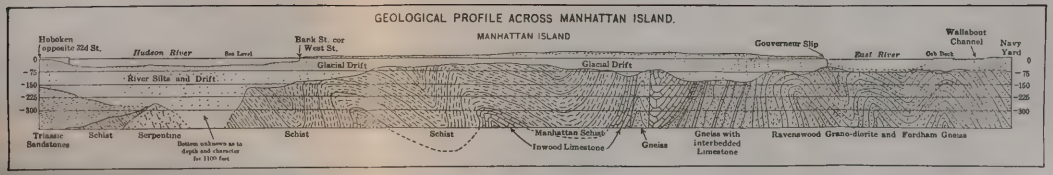
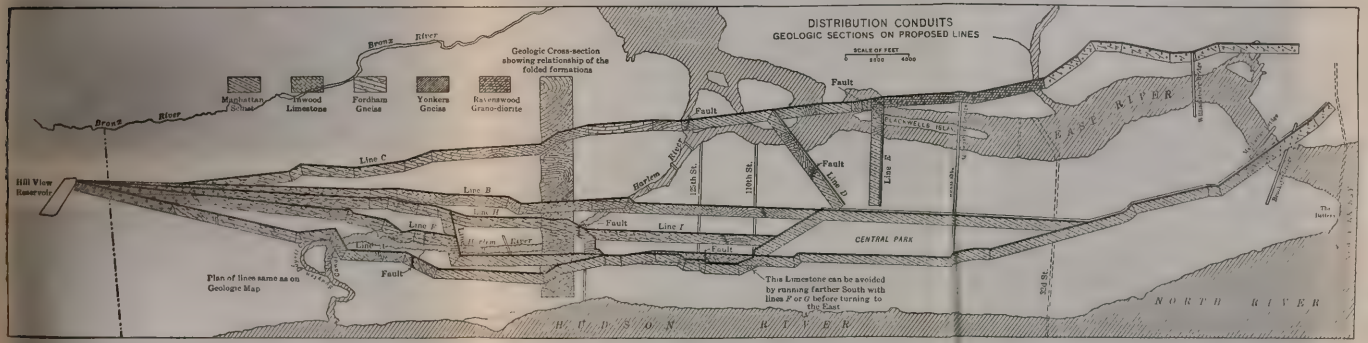
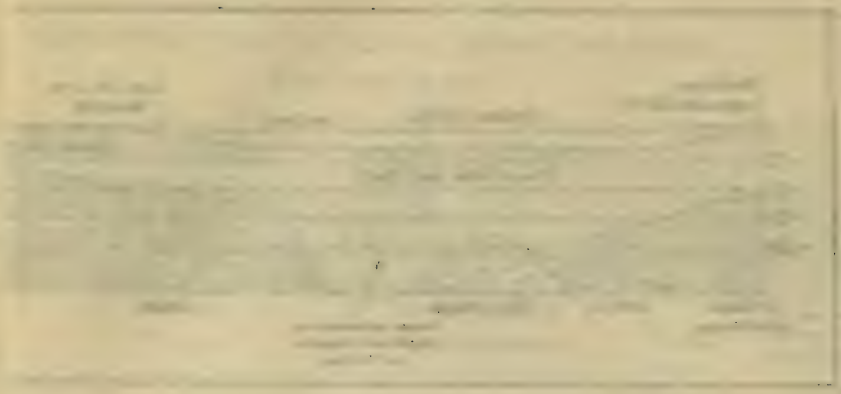
GEOLOGICAL PROFILE ACROSS MANHATTAN ISLAND.







1897.

FIG. 2.—LOWER END OF MANHATTAN ISLAND, WITH TALL BUILDINGS. HUDSON RIVER AT LEFT, EAST RIVER, WITH BRIDGES AT RIGHT.

FIG. 3.—BATTLESHIPS IN HUDSON RIVER: A NIGHT SCENE OF THE ELECTRIC ILLUMINATION. BUILDINGS ALONG RIVERSIDE DRIVE AT EXTREME LEFT.

FIG. 4.—LOWER MANHATTAN AND THE EAST RIVER SUSPENSION BRIDGES BY NIGHT, SHOWING SPECIAL ILLUMINATION OF BRIDGES.

long ago, two pepper-corns a year, is now worth $1 735 000. Within a few years, it is believed, the valuation of the park lands alone would be equivalent to the total funded debt.

By an interesting compilation of figures, someone has shown that, from 1900 to 1910, the net increase of land values in New York City was more than the combined gross output of all the gold and silver mines of the United States by $170 000 000, and greater by $278 000 000 than the aggregate dividends paid during the same 10 years by all the railroads of the United States. Even at such prices, New York real estate, in the main, is considered a profitable investment.

The port of New York is the greatest railroad terminus. New York City is also one of the greatest manufacturing communities; it has 26 000 factories, and the products of its broadly diversified industries have now a value of $2 000 000 000 per year.

Greater New York, created in 1898 by consolidating a number of related cities and towns, comprises five boroughs: Manhattan, occupying the island of that name—"Little Old New York"; The Bronx, on the main land to the north and east; Queens and Brooklyn, on the western extremity of Long Island; and Richmond, which is Staten Island, to the south, at the entrance to the harbor. Numerous small islands are distributed among the boroughs. The total land area is 327 sq. miles, of which 85 000 acres, or 133 sq. miles, are vacant. The water-front extends for 577 miles, including islands and navigable inlets, but none of the New Jersey shore, and the area of the water surface of the harbor is 210 sq. miles.

The City's inhabitants number 5 373 000, as estimated for July 1st, 1913, by the Department of Health; but the whole community directly dependent for existence on the wonderful harbor is far greater, and aggregates approximately 7 000 000 persons. The population is increasing at the rate of 35% each decade. On the average, a baby is born every 6 min. Curves plotted on Fig. 6 indicate the rates of increase of population for the several boroughs and the greater city. Of Greater New York's inhabitants, 12% live in areas having a population density of 400 or more per acre, and 11% in areas having 10 or less per acre. The total of foreign-born is about 2 000 000; of these, 485 000 are from Russia, 340 000 from Italy, 253 000 from Ireland, 280 000 from Germany, 193 000 from Austria, 73 000 from Hungary, 78 000 from England, and 57 000 from Sweden and Norway. In the Boroughs of

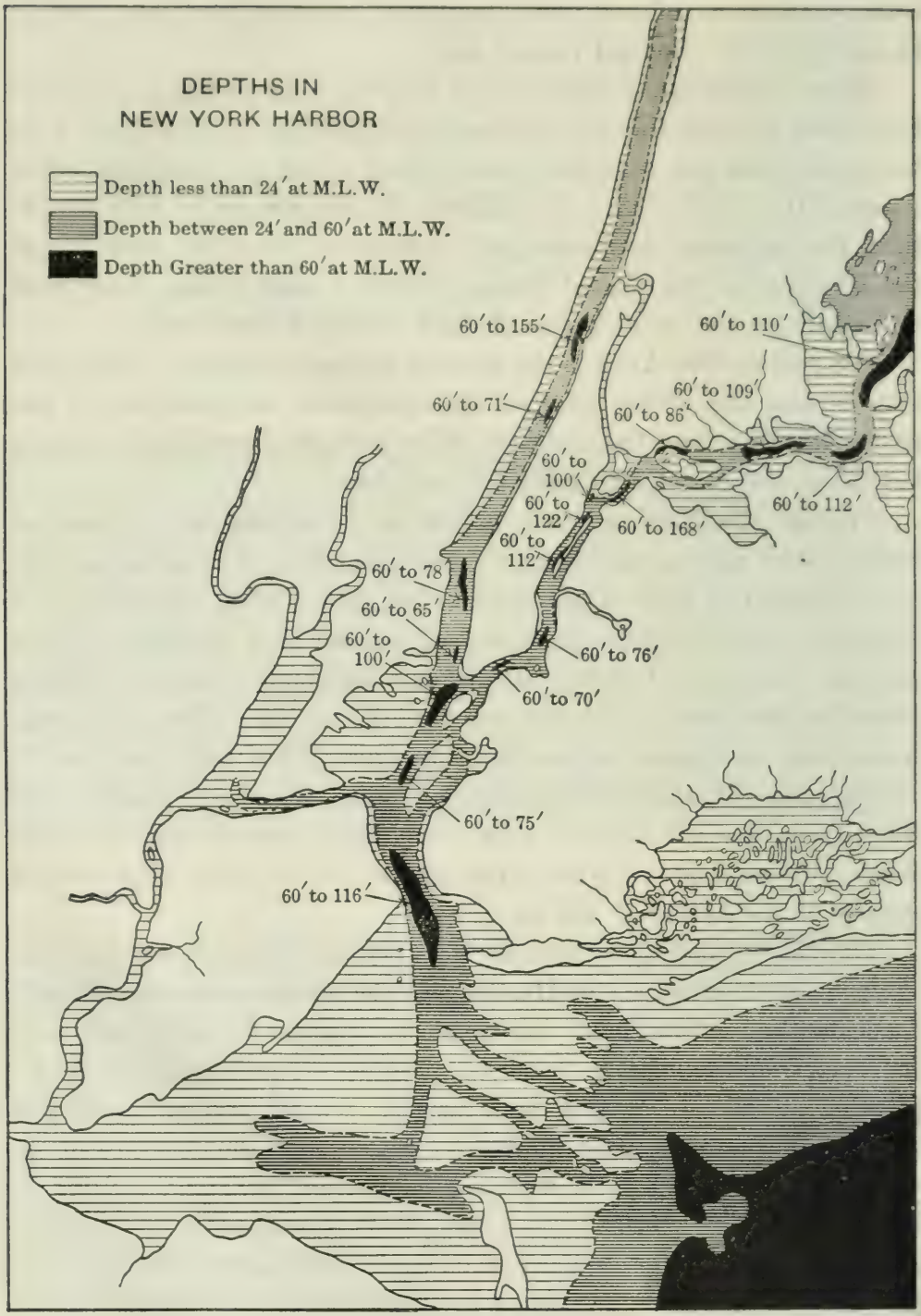

DEPTHS IN
NEW YORK HARBOR
Depth less than 24′ at M.L.W.
Depth between 24′ and 60′ at M.L.W.
Depth Greater than 60′ at M.L.W.
60′ to 155′
60′ to 71′
60′ to 110′
60′ to 109′
60′ to 86′
60′ to 100′
60′ to 122′
60′ to 112′
60′ to 168′
60′ to 112′
60′ to 78′
60′ to 65′
60′ to 100′
60′ to 76′
60′ to 70′
60′ to 75′
60′ to 116′

FIG. 5.

Manhattan and The Bronx 45.4% of the white population are foreign born, and only 15.8% are of native parentage.

Greater New York's increment of population each 2 years is equivalent to a Jersey City or a Kansas City, a Bucharest or a Nottingham, and the annual gain would make an Albany, a Bridgeport, a New Haven, or a Grand Rapids, a Havre or a Geneva. New York's 5-year growth would people a Boston, a Cleveland, a Baltimore, or a Liverpool, and the increase between the last Federal census and the one a decade earlier would have supplied enough inhabitants for a Philadelphia, a Moscow, or a Peking.

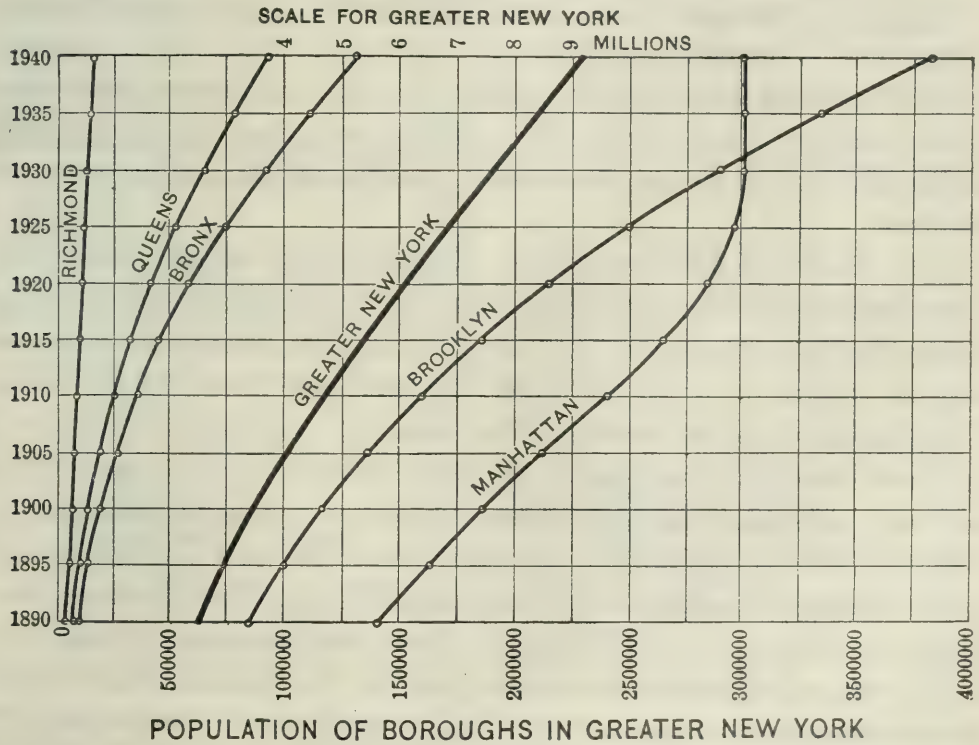


FIG. 6.

In New York there are more than 1 000 000 Jews. The negroes number 121 000, and have increased 10% in a decade. Residents of New York who were born in the Kaiser's empire would populate a Hanover; those from the Czar's domain, an Odessa. The Scandinavians are equivalent to one-sixth of the population of Stockholm. There are enough Italians to fill Turin. Those who once dwelt under the Turkish crescent and in the shadow of the Balkans would fill ten or twelve cities as large as Cettinje. Public utilities administration

and civic government are not a little complicated by these large and diverse foreign elements in the population.

City Government.

New York City is governed under a charter from the State of New York. A Mayor, five Borough Presidents, a Comptroller, the President of the Board of Aldermen, and a few minor city officials are elected once in 4 years, and a board of 73 Aldermen, every 2 years. Plate LIX indicates diagrammatically the general scheme of organization of the City government.

The Board of Estimate and Apportionment, made up of the Mayor, Comptroller, President of the Board of Aldermen, and the five Borough Presidents, is the chief administrative authority of the City; it passes upon all projects requiring expenditure of money, and makes appropriations therefor; it fixes most of the salaries not stipulated by law, and makes the annual budget; it meets in the City Hall weekly, except in summer.

The Board of Aldermen is the legislative body of the City. The Comptroller is at the head of the Finance Department, which audits finally all bills for work, services, and supplies, including pay-rolls and certificates for payments under contracts, and draws orders on the City Chamberlain for the cash disbursements.

Each Borough President is the chief administrative officer of his borough, and appoints and removes heads of bureaus in charge of highways, sewers, topography, and other municipal works. The bureaus are not altogether alike in the different boroughs. Certain functions, better performed for the City as a whole, are entrusted to large departments under commissioners appointed by the Mayor. Some of the most important of these are the Police Department; Department of Water Supply, Gas, and Electricity; Department of Bridges; Department of Docks and Ferries; Department of Public Charities; Law Department; Health Department; Department of Taxes and Assessments; Street Cleaning Department; Bureau of Weights and Measures; Tenement House Department; Fire Department; Park Department; Board of Education, of 46 members, from as many districts; Civil Service Commission; and Department of Correction.

From time to time, for the performance of unusual duties, special commissions have been created by statute. Among these may be men-

PLATE LIX.
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ORGANIZATION OF NEW YORK CITY GOVERNMENT.

BOROUGHS OF GREATER NEW YORK:
Manhattan, Brooklyn, The Bronx, Queens, Richmond.

BOARD OF ESTIMATE AND APPORTIONMENT.

FINANCES, FRANCHISES, PUBLIC IMPROVEMENTS.

Consists of the Mayor, as Chairman, the Comptroller, the President of the Board of Aldermen, and the five Borough Presidents.

Mayor, Comptroller and President of Board of Aldermen, each has three votes; Presidents of Manhattan and Brooklyn, each has two votes; Presidents of Bronx, Queens and Richmond, each has one vote.

CONTROL OF CITY'S EXPENDITURES.

Annual budget is approved by Board of Estimate and Apportionment and then submitted to Aldermen; if latter take no final action in 20 days, budget is deemed adopted as submitted.

Certain special expenditures need previous authorization of Board of Estimate only:

Board of Water Supply, for additional sources, no limit;
Commissioner of Water Supply, Gas, and Electricity, for water supply purposes, not to exceed $2 000 000, in one year;
Rapid transit facilities, no limit;
Docks and ferries (on recommendation of Commissioners of the Sinking Fund), not to exceed $5 000 000, in one year.

BOARD OF ALDERMEN.

(Legislative).

Consists of 73 members elected for 2 years. President of Board is elected as such by the people. Salaries: President, $5 000; members, $2 000.

(The abolition of this board has been recommended frequently.)

CITY CLERK.

Elected by Board of Aldermen for 6 years. Salary, $7 000.

BOROUGH PRESIDENTS:

Elected for 4 years; salaries as below:
have charge of following bureaus:

MANHATTAN.	BROOKLYN.	BRONX.	QUEENS.	RICHMOND.
$7 500.	$7 500.	$7 500.	$5 000.	$5 000.
Highways, Sewers, Buildings, Hall of Records, Public Baths and Public Comfort Stations, Public Bldgs. and Offices. Public Works.	Highways, Sewers, Buildings, Topographical, Encumbrances and Permits, Public Bldgs. and Offices.	Design, Topographical, Highways, Sewers, Buildings, Public Bldgs. and Offices.	Highways, Sewers, Buildings, Topographical, Street Cleaning.	Highways, Sewers, Buildings, Public Works, Engineering Construction, Topographical, Street Cleaning, Public Bldgs. and Offices.

THE MAYOR.

Chief Executive. Elected for 4 years. Salary, $15 000. Is Chairman of Board of Estimate and Apportionment. Appoints heads of city departments.

Police Department.
Department of Health.
Fire Department.
Department of Street Cleaning.
Department of Water Supply, Gas, and Electricity.
Law Department.
Department of Parks.
Department of Docks and Ferries.
Department of Charities.
Department of Correction.
Bellevue and Allied Hospitals.
Department of Buildings
Department of Bridges.
Tenement House Department.
Department of Taxes and Assessments.
Bureau of Licenses.
Bureau of Weights and Measures.
City Magistrates.
City Chamberlain.
Commissioners of Accounts.
Civil Service Commission.

The Mayor also appoints members of following Boards:
Assessors, Armory, Education, City College, Normal College, Art Commission, Board of Water Supply.

COMPTROLLER.

Chief of Finance Department.
Elected for 4 years. Salary, $15 000.

Bureau of Collection of Taxes.
Bureau of Miscellaneous Revenue.
Bureau of Assessments and Arrears.
Bureau of Audit of Claims.
Bureau of Investigation and Statistics.
City Payments.
Stock and Bond Division, etc.

COMMISSIONERS OF THE SINKING FUND:

The Mayor, Comptroller, City Chamberlain, President of Board of Aldermen, Chairman of Finance Committee of Aldermen.

Has charge of sale and leasing of all public property, including real and personal estate, and of the investment of moneys which constitute the sinking fund for the redemption of the City's debt.

PUBLIC SERVICE COMMISSION OF THE FIRST DISTRICT.

(A State Board appointed by the Governor)
Five Commissioners, at Salary of $15 000.

Has supervision in Greater New York of matters relating to public utilities, such as railroads, street railways, gas and electricity; and is charged with construction and operation of rapid transit facilities. All expenses are paid by the City.

tioned The Aqueduct Commission (1883 to 1910), which constructed the New Croton Aqueduct and most of its appurtenant reservoirs; The Rapid Transit Railroad Commission (1894 to 1907), which built the first subway and was superseded a few years ago by the State organization known as the Public Service Commission of the First District, in charge of the very extensive system of subway and elevated railways now in course of construction, as well as many other important matters; and The Board of Water Supply (created in 1905), which is building the Catskill Aqueduct and reservoirs for obtaining an additional public water supply.

Engineer corps, with chief and consulting engineers at their heads, are maintained by the several boroughs and such departments and commissions as have charge of the construction and maintenance of public works. The Chief Engineer of the Board of Estimate and Apportionment reports on all public works projects brought before that board, and hence occupies the position corresponding most closely to that of city engineer in many cities, although he has no construction in charge. In the office of this Board there is a Franchise Bureau, the engineers of which examine proposed franchises for public utilities, and devote some attention to existing franchises to see that the terms are complied with.

In the Comptroller's office there is a small corps of engineers whose duty it is to examine periodically during progress of construction all work of an engineering character being done for the City, in order to determine, before partial and final payments are approved, whether the contract requirements have been satisfied. These examinations are made rapidly, and are in addition to the detailed supervision and inspection by the engineers of the department conducting the work.

Altogether, New York employs 2 000 engineers and engineering assistants of various ranks and kinds; the total number of regular city employees is 85 000, and in addition, 22 000 are temporarily engaged, principally in connection with elections and other occasional work. The annual pay-roll amounts to $89 000 000. The City's annual budget is approximately $200 000 000, in addition to which $80 000 000 are spent otherwise for public purposes each year, such as additional water supply works, rapid transit railroad subways, street improvements, and docks. Practically all employees are under a civil service

system, controlled by entrance and promotion examinations, and directed by a Municipal Civil Service Commission.

The receipts of the City Treasury in 1912 were $524 000 000—$154 000 000 from taxes, water rents, and similar items; $334 000 000 from the sale of securities; $10 000 000 from assessments; $5 700 000 from the liquor tax; and $1 800 000 from the rapid transit contracts. The City will pay in 1913 for interest and sinking fund charges on its debt $55 000 000. The Department of Education, in 1912, spent $39 000 000. The City spent $24 000 000 that year for the new Catskill Water Supply System, and invested $14 000 000 in subway construction. It cost New York $9 000 000, in 1912, to clean its streets, $10 000 000 to put out fires, and for police protection $17 000 000.

For governmental purposes, Greater New York's territory is also divided into four counties: New York, Kings, Queens, and Richmond, and the creation of a fifth, Bronx County, was referred last fall to the voters of that portion of the City. Bronx County will elect officers some time in the year 1913, and these officers will assume their duties on January 1st, 1914. The counties have to do principally with certain courts, records of real estate transfers, and kindred matters.

The National Government maintains within the city limits a custom house, an immigration station, federal courts, several post offices and numerous branches, a navy yard, an army departmental headquarters, and a number of fortifications. The New York post office does an annual business of nearly $600 000 000. The world-famous West Point Military Academy, founded in 1794, is 55 miles up the Hudson River, and is reached by a delightful sail; here the Government is spending millions of dollars on beautiful new buildings.

City Map.

Probably the earliest demand for an official map of the City or its parts arose in connection with assessing real estate for purposes of taxation. Ward maps are mentioned in records of the city government nearly a century ago. A law passed in 1857 established the methods of taxing real estate which are still in use, and directed that the tax commissioners should appoint a surveyor from among the City Surveyors to make surveys, correct the ward maps, and prepare new maps. One of the oldest official maps is the Randall map, giving a street plan south of 155th Street; it was authorized by the Legislature

in 1807, and filed in 1811. There are, however, older private maps extant.

A law enacted in 1892 required the Commissioners of Taxes and Assessments to prepare a "block map of taxes and assessments," with triplicate copies, the original to be filed in the office of the Commissioners, one copy to be for the Deputy Tax Commissioners, one for the Bureau of Arrears of the Finance Department, and the third for the Bureau of the Water Register. These block maps, styled the permanent tax map, have been completed for all portions of the greater city having established street systems; they comprehend all Manhattan and Brooklyn, and that portion of the Bronx Borough west of the Bronx River. For the remainder of the City, the map is known as the tentative tax map. The permanent tax maps are drawn to a scale of 50 ft. to 1 in., the tentative tax maps to a scale of 80 ft. to 1 in., and smaller scales, on 14 by 22-in. sheets, and bound in volumes containing 35 sheets, together with an index and title sheet.

Land value maps, 142 in number, are published annually by the Department of Taxes and Assessments. These maps cover the whole City, and are for the following purposes: (1) to aid in assessment by giving each assessor a comprehensive view of his territory, with comparable figures on every street; (2) to assist the Commissioners in passing upon petitions for reductions of assessed valuations; (3) to enable the public to judge of the fairness of the assessor's work, and to aid the department by suggestions.

From 1652 to 1874 New York City had an area of 22 sq. miles. In 1874, 20 sq. miles were added; in 1895, 20 sq. miles more; in 1898, 260 sq. miles. The present area is sixteen times the original. In 1912, 523 330 parcels of real estate were assessed.

Topographical maps, based on careful surveys, have also been prepared for Manhattan, Brooklyn, and The Bronx; those for the Boroughs of Queens and Richmond are in progress, the surveys having been very nearly completed, the maps for Queens about half drawn, and those for Richmond well started. There are few good maps of the numerous pipes, ducts, and other structures buried beneath the streets. Recently, however, the Borough of Brooklyn established a Bureau of Sub-surface Structures, the engineers of which have made a creditable beginning in compiling maps showing all structures under the streets of that borough. Other boroughs also are directing attention to this

sorely needed set of records. Unfortunately, much of the necessary information is lacking, and can be obtained only from excavations for new work or repairs made from time to time or from test trenches dug across the streets.

Harbor Channels and Freight Terminals.

The Lower Bay is comparatively shallow, so that, for modern commerce, artificial channels are necessary. From time to time, for many years, the Federal Government has improved New York's harbor by deepening and widening existing channels and dredging new ones. It is now possible for vessels of the deepest draft (40 ft.) to enter or leave the harbor and to be berthed at all stages of tide. The tidal range is moderate, averaging only 5 ft. It ranges from 4.4 ft. at the southern tip of Manhattan to 7.2 ft. in the East River near Long Island Sound. Compare these figures with 9.6 ft. in Boston Harbor, 11.5 ft. at Antwerp, 17.6 ft. at London, and 21.3 ft. at Liverpool. In total, the Government has expended $18 500 000 on harbor improvements at the Port of New York.

From 25 to 35 years ago, few American harbor engineering enterprises attracted more popular attention than did the Hell Gate improvement in New York Harbor. Rock reefs, which were very dangerous to vessels, obstructed the East River near its connection with Long Island Sound. Through these narrow, tortuous passages the tidal flow rushed with high velocities and very troublesome cross-currents and eddies, which gave the place its original name, "Whorl Gate." On September 24th, 1876, in 7 426 lin. ft. of tunnels in one of these rocks, Hallets Point, was fired the largest single charge of high explosives, 48 000 lb., which had been attempted to that date. Work was continued intermittently by various methods. On October 10th, 1885, Flood Rock, of 9 acres area, was partly demolished by the explosion of nearly 300 000 lb. of dynamite and rackarock in 21 670 lin. ft. of tunnels, from which 80 000 cu. yd. of rock had been excavated. Subsequent to the blast, the shattered rock was removed by dredging, continued for 20 years, and completed by recent drilling and blasting from scows and the removal of the loosened rock by a powerful dipper-dredge. By these operations the channel has been rendered safe and given a depth of 26 ft. for a width of 1 200 ft.

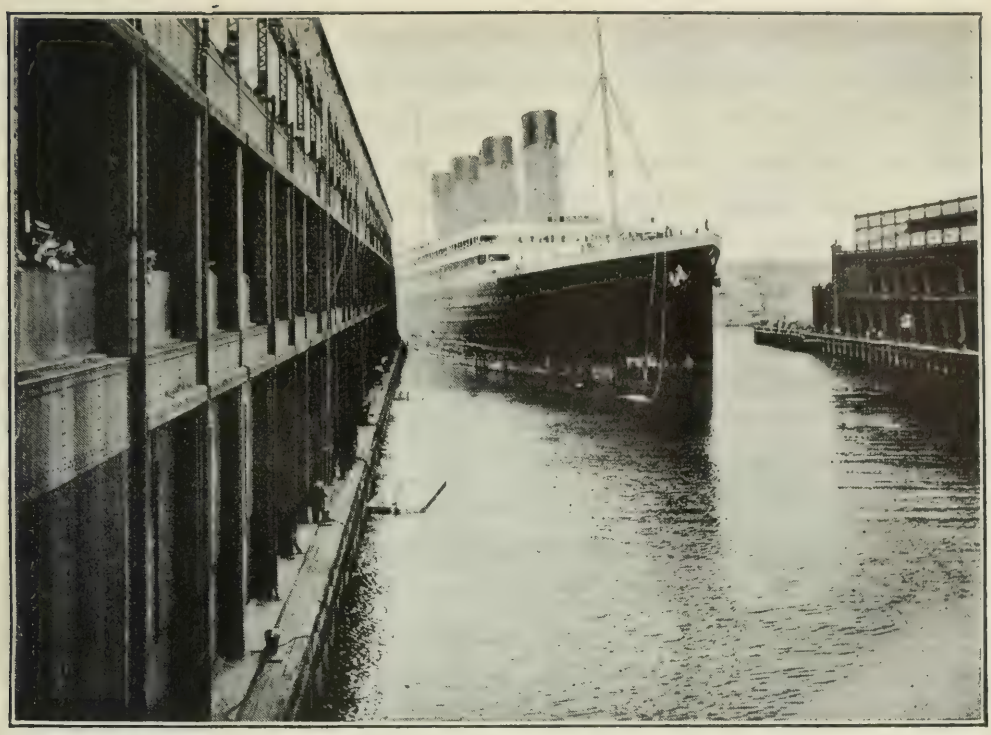

FIG. 7.—CHELSEA STEAMSHIP TERMINAL. THE "OLYMPIC" WEARING AND WARPING INTO SLIP, USING NORTHWEST CORNER OF PIER AS A FULCRUM IN THE MANEUVER. OBSERVE FLOATING FENDERS WHICH ASSIST IN BREASTING THE VESSEL OFF THE PIER. STEAMSHIPS ALWAYS DOCK WITH BOW INSHORE.

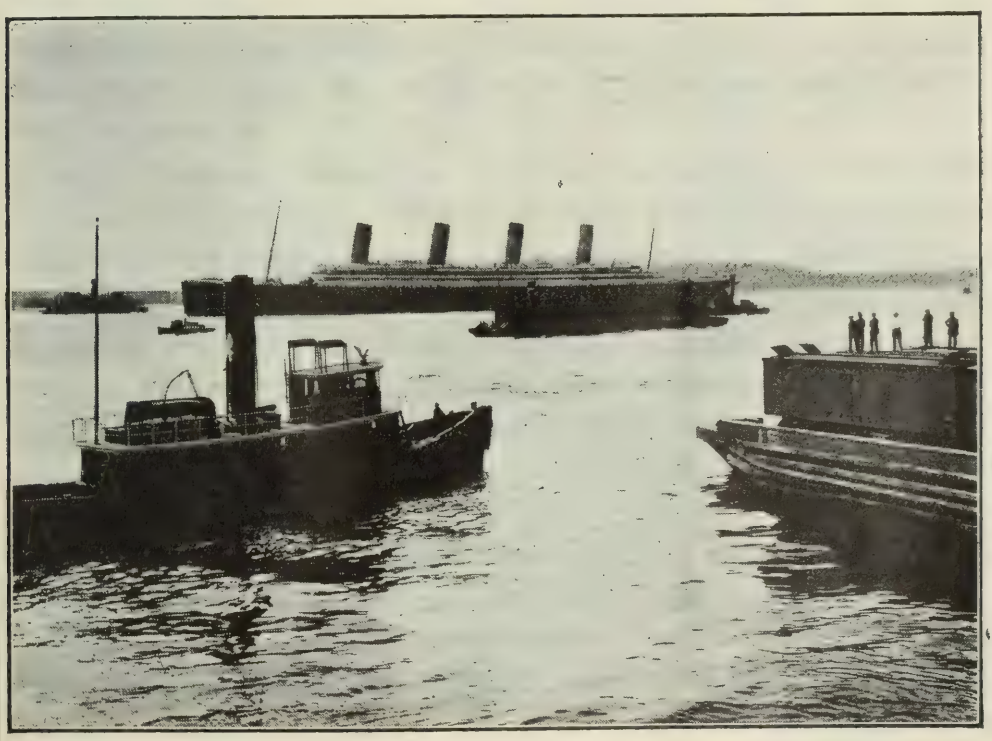

FIG. 8.—THE "OLYMPIC" IN MID-STREAM, NORTH RIVER; ALSO FERRYBOATS, FREIGHT-CAR FLOATS, AND STEAM TUGS.

Of wholly different character has been the Ambrose Channel improvement, at the southerly entrance to the harbor, begun in 1899 and carried on partly by contract and partly by Government plant. By suction-dredging 50 000 000 cu. yd. of sand, gravel, and mud, a channel, with 40 ft. depth at low water, 2 000 ft. wide and 7 miles long, has been created. The sea-going dredges, used exclusively, deposited the excavated material in deep water several miles out in the ocean. On this work the Government has used four specially built dredging steamers, each equipped with two 20-in. centrifugal pumps and each capable of discharging into the steamer's bins, an average of 6½ cu. yd. of hard-packed fine sand or 10 cu. yd. of loose sand and fine gravel per minute. The average liquid capacity of each pump is 86 cu. yd. per min. An average total cost as low as about 6 cents per cu. yd. of solid material, has been attained by the U. S. Engineers in charge. The contract price was 9 cents, and the contractor finally abandoned the work. In round figures, the total cost of improving this entrance to the harbor, formerly known as the East Channel, has been $5 000 000.

Private corporations have constructed large plants for warehousing merchandise and effecting transhipment economically from vessel to railroad. Attention is directed to the Bush Terminal and the New York Dock Company's plant in South Brooklyn. Very much more must be done along these lines, publicly or privately, in the immediate future, to meet the needs, which have already outstripped the facilities provided.

Bush Terminal, created by a private corporation, on the South Brooklyn water-front, is a complete, yet constantly growing, community, engaged in the shipping, warehousing, manufacturing, and distributing of a great variety of merchandise. It occupies 200 acres of land, on which are nine very large industrial buildings, with others in progress of construction. Ultimately, there will be thirty such buildings. There are also 130 warehouses. Most of the industrial buildings are 700 ft. long, 75 ft. wide, and 6 stories high, of reinforced concrete, unburnable construction, and designed for floor loads of 200 lb. per sq. ft. Throughout these buildings run pipes and wires supplying water under pressure, gas, live steam at 100-lb. pressure, and direct-current electricity at 220 volts. A modern sprinkler system, which alone has cost nearly $1 000 000, ramifies through all the buildings and assures the prompt extinguishment of an incipient fire in

the contents of any portion of the premises. One of the industrial tenants is a large paper manufactory, said to be the only one at tidewater. This paper plant has a capacity of 100 tons per day. Seven modern piers, each about 1 300 ft. long, accommodate steamship lines which send vessels to all parts of the world. The railroad yard can store 2 000 cars. The terminal has its own locomotives, tugs, lighters, and motor trucks; its post office, telegraph office, and express station. The economy and other advantages of such adjustment of commercial functions and the means for performing them are self-evident.

Docks and Ferries.

To accommodate the vast commerce which seeks the port, there are about 150 large privately-owned piers on Manhattan Island alone, and 850 piers in all, of which the City owns 230; there are many more on the New Jersey shore, where several great European steamship lines have their termini. The City has recently constructed nine 800-ft. piers immediately below 23d Street, on the Hudson shore of Manhattan, most of which are 125 ft. wide, with double-decked steel pier and bulkhead sheds. The piers are of wooden piles, with creosoted plank decks having concrete covering. The street façades of the pier sheds are of reinforced concrete, and the steel columns in them have jacking girders at their bottoms so that settlement may be compensated by using jacks and placing shims between the column bases and their masonry pedestals. These Chelsea piers accommodate some of the greatest transatlantic liners, but were hardly completed before the building of 900-ft. steamers showed them to be inadequate; consequently, two have been extended temporarily to 925 ft. Eight new piers are nearing completion in South Brooklyn.

For the development of a steamship and railroad terminal and an extensive system of docks at Jamaica Bay, on the Long Island shore, the City has appropriated $1 000 000 to begin the dredging of the bay, and the Federal Government has started the deepening of the entrance. The dredged materials are to be deposited back of the bulkhead lines.

Great projects are being studied for other improvements on the Long Island shore; for larger piers on the middle western shore of Manhattan, and for the better organization of the port as a whole, so as to facilitate the transfer of merchandise between ships and rail-

FIG. 9.—BUSH TERMINAL: VIEW FROM BALLOON AT AN ELEVATION OF 1 500 FEET.

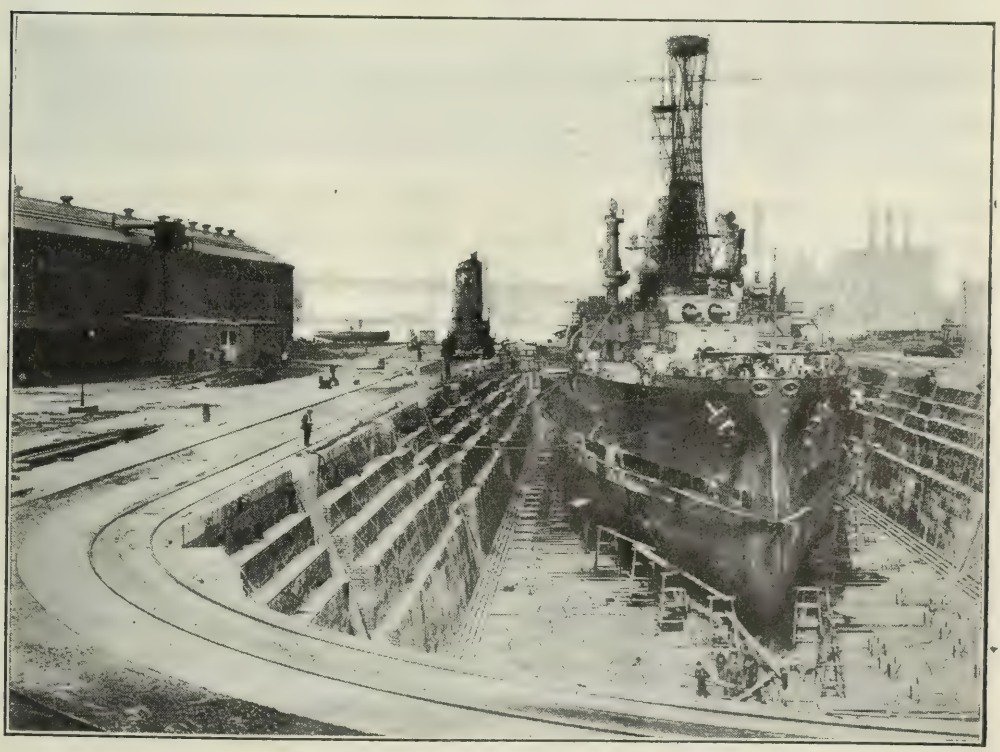

FIG. 10.—BROOKLYN NAVY YARD: DRY DOCK NO. 4, WITH BATTLESHIP "UTAH". FORTY-TON LOCOMOTIVE CRANE AT LEFT. WILLIAMSBURGH BRIDGE IN BACKGROUND.

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roads and warehouses. For the new Manhattan piers, lengths up to 1 200 or even 1 500 ft. are being discussed, and the estimates of cost range from $15 000 000 to $25 000 000. Owing to the opposition of the Army Engineers to the moving of the pier-head line farther from the shore, a portion of the length of the proposed piers may have to be obtained by excavating the slips into the land. If the scheme is carried out on these lines, this will involve great expense for the acquisition of property, and much of the excavation will be in rock, if some of the locations suggested are adopted. Quays and diagonal piers have been suggested, in lieu of piers extending perpendicularly from the shore line. Several schemes for an elevated, surface, or underground marginal railroad along the western water-front of Manhattan are receiving serious consideration. Such a system of tracks is intended to inter-connect all the railroad and steamship lines entering this part of the City, and to serve the warehouses and other commercial establishments in the vicinity so as to curtail cartage and other handling charges and reduce the time required for freight movements. Connection has been proposed, by means of under-river tunnels exclusively for freight, to great railroad car classification yards on the New Jersey shore and the meadows back of the Palisades.

There are 72 public piers and bulkheads for the accommodation of miscellaneous water-front business. Pier leases yield the City a net annual revenue of $3 000 000. Recreation piers, of which there are eight on Manhattan alone, afford play places and better air for thousands in the congested sections. The one shown in Fig. 12 is 722 ft. long, 50 ft. wide, and is typical in its construction.

The New York, or Brooklyn, Navy Yard is the most important in the country. It contains extensive plants for building and equipping naval vessels and making naval repairs of all sorts. This yard has four dry docks, one of timber, one of timber and concrete, one of masonry, and the fourth, recently completed, of concrete. Dry Dock No. 4 is notable for being the largest on this side of the Atlantic, and for the foundation difficulties overcome. After the failure of methods tried by two contractors, a third completed the undertaking successfully by the aid of pneumatic caissons. As the years passed in these endeavors, the design of the dock was improved, and its size was materially increased to meet the constantly growing sizes of vessels.

As now completed, the dock is 700 ft. long, 110 ft. wide at the entrance, and has 35 ft. of water over the sill. It is wide enough to accommodate any vessel which can pass the Panama Canal locks, and any not exceeding 685 ft. in length. It cannot now receive the greatest modern commercial steamships, but if the building containing the central power plant of the navy yard should be moved, the dock can readily be lengthened.

Erie Basin, on the South Brooklyn water-front, is widely known to ship owners. Around this body of water are collected many establishments engaged in ship building, repairing, and outfitting, and here are several noteworthy floating dry docks. One recently completed has a length of 425 ft. and a lifting capacity of 7 000 tons; it is 100 ft. wide over all, and can receive vessels of 21 ft. draft. Its bottom platform and pontoons are of timber, and it has hollow side-walls of steel. Having an ample pumping equipment, it has raised vessels of its full capacity in 35 min., and smaller ones in from 15 to 20 min.

For protecting the shores and reclaiming lands partly under water, many miles of bulkhead or sea wall have been built, and this work is constantly being extended. Two principal types are used: one where there is rock foundation at reasonable depth, the other where the bottom is soft. The latter is known as the mud-flotation type. This wall is of comparatively light section, and supported on vertical piles. Other vertical piles are driven back of the wall to support a platform just above mean low water, on which is carried the few feet of filling. Heavy rip-rap is deposited among the piles, to a considerable depth below the bottom of the wall, and spreads to a wide base beyond the toe of the wall and back of the platform piles. Inclined piles are driven from the top of the rear platform piles to the bottom of the wall piles. By this construction, the wall is materially relieved from lateral thrust. For founding the rock type of wall, concrete in bags is first deposited under water to level up the inequalities of the rock before placing the lowest course of concrete blocks. For the construction of both types of walls concrete blocks, weighing from 30 to 90 tons, fabricated in yards on the water-front, are transported and set with the aid of a 100-ton floating derrick. Thousands of acres of land have been reclaimed and protected by these walls.

Scores of ferry-boats are run by the railroads and other corporations, principally between Manhattan and the other shores of the

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FIG. 11.—THE CHELSEA STEAMSHIP TERMINAL, NORTH RIVER WATER-FRONT, MANHATTAN.

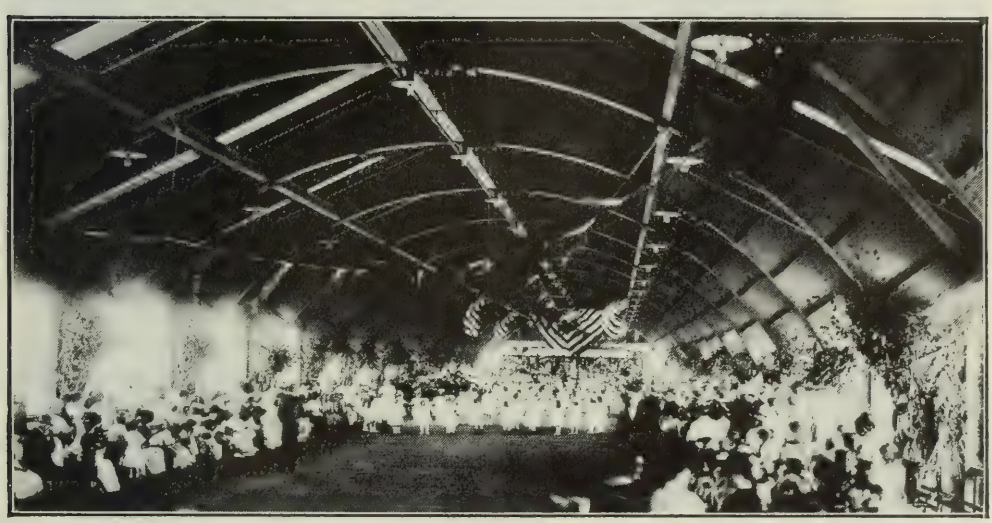

FIG. 12.—MUNICIPAL RECREATION PIER: INTERIOR OF UPPER DECK, SHOWING CHILDREN DANCING.

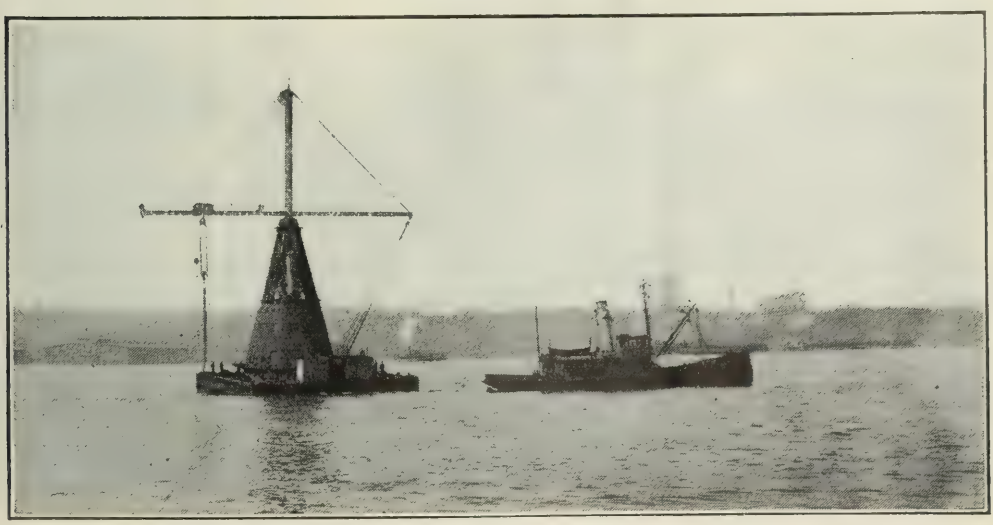

FIG. 13.—THE 100-TON FLOATING DERRICK OF THE DEPARTMENT OF DOCKS AND FERRIES.

FIG. 14.—MUNICIPAL FERRIES: WATER-FRONT OF SOUTH FERRY HOUSE, AT LOWER EXTREMITY OF MANHATTAN ISLAND.

FIG. 15.—MUNICIPAL FERRYBOAT PLYING BETWEEN SOUTHERN END OF MANHATTAN ISLAND AND STATEN ISLAND.

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PLATE LX.
TRANS. AM. SOC. CIV. ENGRS
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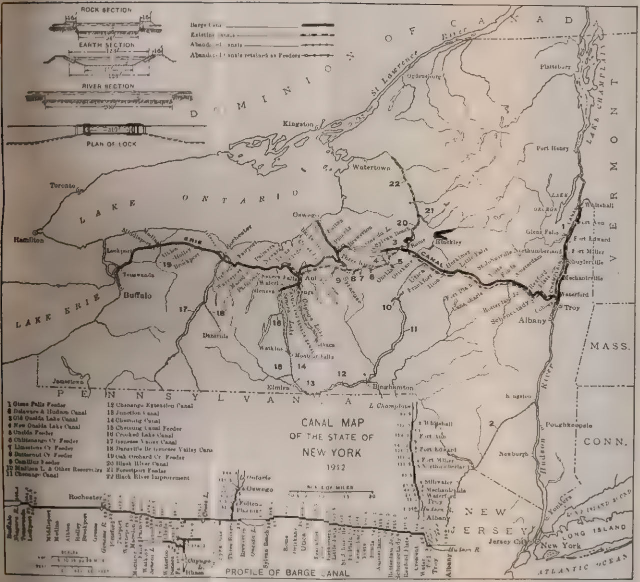
OF NEW YORK CITY.



port, and the City operates ten boats from the south extremity of Manhattan to Staten Island and Brooklyn. Some of these are among the largest and swiftest ferry-boats afloat, and all the modern ones have screw propellers. Ferry rides from Cortlandt Street to Weehawken (West Shore Railroad), from South Ferry to Staten Island, from 42d Street, Manhattan, to Broadway, Brooklyn, and from 99th Street, Manhattan, to College Point, in Queens Borough, afford good views of the water-fronts and harbor. One of the common sights is a pair of huge car floats, each containing 23 freight cars, with a steam tugboat between, propelling them from one steamship or railroad terminus to another.

Barge Canal.

To provide water transportation across New York State, for bulk freight principally, a system of canals is being constructed to replace and extend the Erie Canal, dug during the period from 1817 to 1825, and the Oswego and Champlain Canals. It is known as the Barge Canal, and will cost $110 000 000. Fourteen terminals, for the accommodation of the canal-boats which will come down the Hudson, are to be built in the various boroughs of New York City where they will be convenient for the local distribution of goods or for transhipment. There are to be thirteen canal ports besides New York City, and the total cost of terminals is put at $20 000 000. Lack of such terminals was a principal cause of the commercial failure of the canals in recent years. Aggregating 445 miles in length, these canals will have a usual depth of 12 ft.; width at bottom, 75 to 94 ft.; and 57 locks large enough to pass 3 000-ton barges. It is anticipated, however, that barges having a capacity of 800 to 1 500 tons will be more commonly used. The completion of these canals is expected in 1915.

The Barge Canal has been located so as to follow the natural waterways much more closely than the old canals, and therefore, in large measure, it is a river canalization. Besides the constructed canals, the system embraces 350 miles of river and lake navigation. When this project is completed, there will be within the State 1 050 miles of such waterways, and along them will be dwelling 90% of the State's population. In addition, the Canadian and Vermont frontiers of the State have 500 miles of lake and river navigation, and the Hudson River affords 150 miles more, making a grand total of 1 700 miles.

Railroads.

Ten great railroad systems, each combining a number of important railroads, terminate at New York Harbor. Until recently, most of them have been able to deliver passengers and merchandise on Manhattan Island only by ferry. For a long time the New York Central and the New York, New Haven and Hartford Railroads have approached from the north and northeast by bridges. To accommodate the greatly increased traffic of these two systems, the Grand Central Station, on 42d Street, is being rebuilt of much greater size, with extensive changes in tracks and streets, involving a total expenditure of $140 000 000, in round numbers, exclusive of buildings not used directly for railroad purposes.

Grand Central Terminal.

To devise a terminal where space is practically unlimited, is a simple proposition; but to accommodate in a congested metropolitan center 5 000 000 residents and several more millions of commuters and miscellaneous travelers, is not simple. The four tracks of the New Haven Railroad, the two tracks of the Harlem Railroad, and the four tracks of the Hudson River branches of the New York Central Railroad converge into Park Avenue, and their traffic must be absorbed at or near 42d Street. The results of the efforts to solve this problem in the most congested district of the country, where property values approach the fabulous, are shown in Figs. 16 to 21.

The main track level occupies nearly the entire area between 42d and 50th Streets and Madison and Lexington Avenues, with a loop at the south end, around which equipment may move. This level has 19 miles of track, with 29 tracks adjacent to platforms. In-bound trains in Park Avenue are diverted at either 57th or 52d Streets to the five westerly tracks connected with the loop. Passengers are unloaded on these platforms, and the train then passes around the loop to the storage tracks on the east side of the yard. Out-bound trains leave from the stub tracks in the center of the yard.

The suburban level extends from 42d to 52d Street, with a maximum width of 707 ft. It also has a loop for turning, and the method of operation is similar to that for the main-track level. The suburban level has about 15 miles of track and 17 platforms. The total area of the two levels is 79 acres.

16 81

FIG. 16.—GRAND CENTRAL TERMINAL: A VIEW SOUTHWARD IN 1906, OVER THE OLD YARD IMMEDIATELY NORTH OF THE STATION, WHICH IS IN THE CENTRAL BACKGROUND.

FIG. 17.—GRAND CENTRAL TERMINAL: EXCAVATION FOR STATION. LOOKING SOUTH FROM 44TH STREET AND VANDERBILT AVENUE, MAY, 1911.

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FIG. 18.—GRAND CENTRAL TERMINAL: EXCAVATING FOR THE NEW TRACK LEVEL, IN ROCK, CLOSE TO PRIVATE BUILDINGS, NEAR 50TH STREET. AUGUST, 1906.

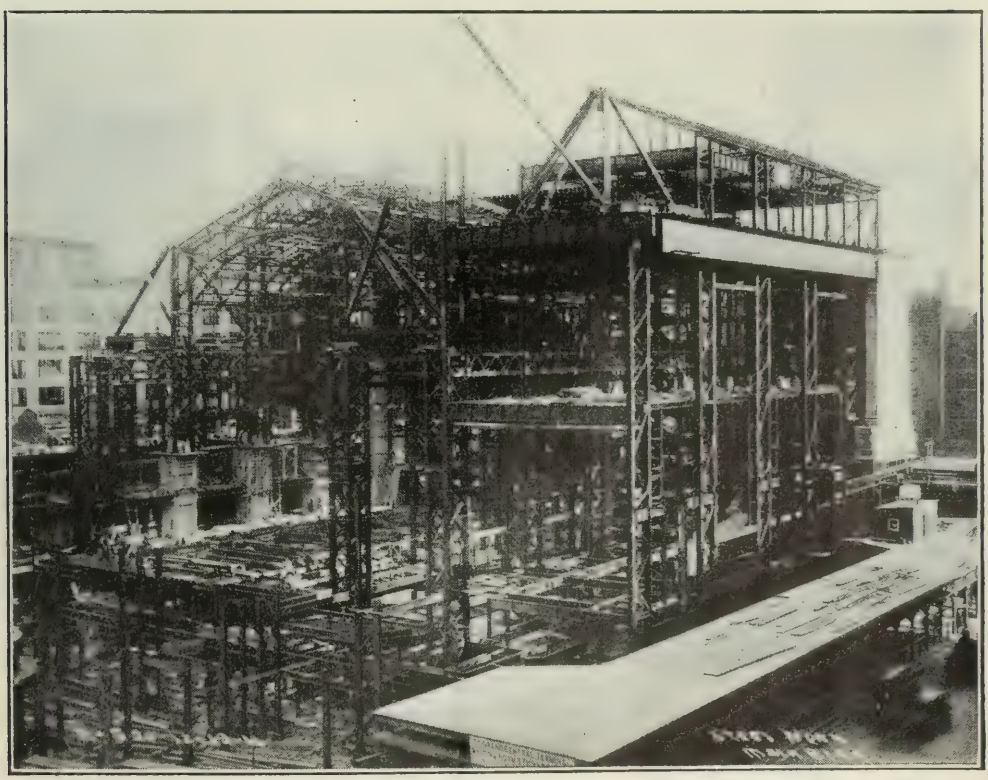

FIG. 19.—GRAND CENTRAL TERMINAL: STEEL FRAME OF THE NEW STATION IN PROCESS OF ERECTION, DECEMBER, 1911.

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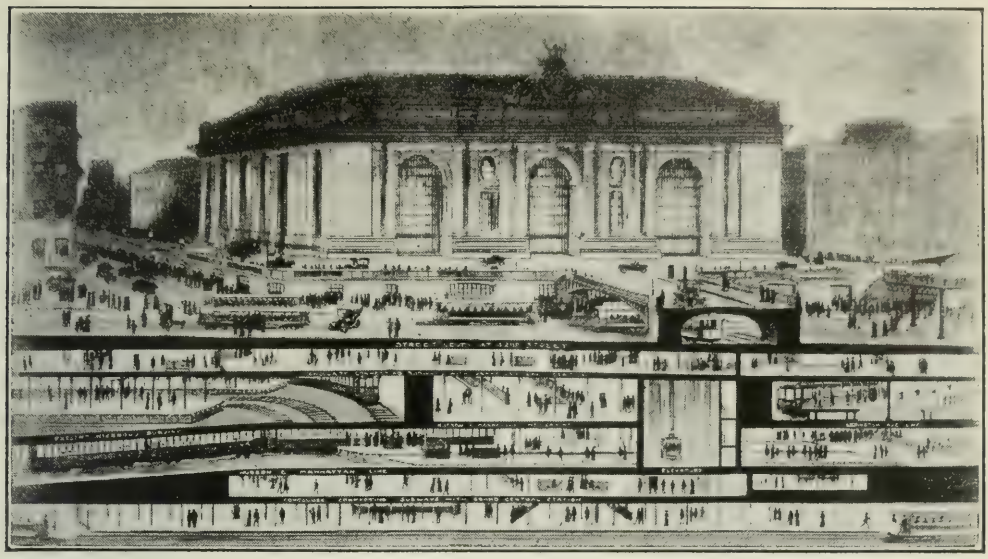

FIG. 20.—GRAND CENTRAL TERMINAL: SEVEN LEVELS OF ELEVATED, SURFACE, AND SUB-SURFACE TRAVEL. THE BRIDGE CARRIES PARK AVENUE OVER 42D STREET.

FIG. 21.—GRAND CENTRAL TERMINAL: THE NEW STATION AND THE BRIDGE CARRYING FOURTH AVENUE OVER 42D STREET. BILTMORE HOTEL (AT THE LEFT), AND OTHER BUILDINGS (LEFT AND RIGHT) ERECTED BY THE RAILROAD COMPANY OVER THE TRACKS.

If this terminal were worked to its maximum capacity for 24 hours, using it purely as a station, and providing holding capacity for equipment at outlying points, it could take care of about 3 300 in- and outbound trains, or approximately 2 300 000 people.

To grasp fully what is being accomplished one must recall what formerly existed. Here, on the street level, was a maze of tracks, over which smoking engines and noisy trains shuttled by day and night. Crossing this area on the lines of intersecting streets were elevated footbridges with stairways at each end. The whole spectacle was an eye-sore. The introduction of electricity did away with the smoke, but the ugly yards remained. Park Avenue ended at 42d Street and did not reappear as a thoroughfare until 50th Street was reached.

The New York Central Railroad owns the air rights of all the property comprising the yards, hence came a plan by which the main station was to be the beginning of a magnificent group of buildings, of uniform architecture, which would extend over all the tracks, not only hiding them, but adding to the attractiveness and usefulness of the whole section. In addition, all the cross-streets from 45th to 50th Streets, some of which had been carried over the old yard with only narrow footbridges, have become full-sized streets, and Park Avenue is to become an unbroken boulevard, thus greatly relieving overcrowded Fifth and Madison Avenues.

The elevated street around the main station is a continuation of Park Avenue, which is soon to be carried over 42d Street on an arched viaduct. This viaduct will be constructed by the City. In addition to the main station, which occupies the center of the group, there are now completed or under construction: the Grand Central Palace, the Railroad Company's Office Building, the Biltmore Hotel, the Adams Express Building, the Railroad Young Men's Christian Association, the Yale Club, and the United Cigar Stores Building. As the development proceeds, hotels, places of amusement, apartment houses, and other edifices will take their places in the group. The architecture has something of the Doric motive, but approaches more closely the Tuscan, modified by the French Renaissance.

The "Grand Central Palace" is a large, twelve-story loft building occupied by the Association of Wholesale Furniture Dealers. This building, costing $2 000 000, was erected over a continuous stream of in- and out-bound trains, without disturbance of the schedule, in 8

months and 18 days; the steel frame was erected at night (between 11 P. M. and 7 A. M.) and riveted during the day.

In Fig. 17 are shown the excavating operations in the rock on the site of the station building. There have already been used nearly 800 000 lb. of dynamite. The largest single item of work is the excavation, requiring the removal of approximately 3 100 000 cu. yd. of material, of which about two-thirds are rock. Earth and rock from the excavations are utilized for widening the roadbed for additional tracks along the Hudson River and on the Harlem Division. Much of the excavating has to be done near buildings owned by other parties. The general character of this work is indicated in one of the illustrations. In this particular case the building was underpinned for a depth of 30 ft., the excavation in front of it being 45 ft. deep.

A steel deck, over the suburban level station, supports the express level tracks, which cover an area of about 38 acres, with provision for building columns to be inserted later. Above the express level there are about 1¾ miles of viaducts. The main station building contains approximately 29 000 tons of steel. Provision is made in this structure for carrying the central portion up to a height of 20 stories. In the entire terminal, including tracks, viaducts, and buildings now definitely determined upon, there are approximately 150 000 tons of steel. Portions of the new terminal have been in use for some time. It is expected that within a year most of the work will be completed.

Pennsylvania Terminal.

A great system of tunnels, aggregating 13.7 miles in length, with a magnificent passenger station between Seventh and Ninth Avenues, 31st and 33d Streets, was completed in 1910 by the Pennsylvania and Long Island Railroad systems. These tunnels start far back from the New Jersey shore. Two single-track tubes pass beneath the Hudson River and as far as the station yard; two double-track tunnels cross Manhattan Island, and four single-track tubes pass beneath the East River and to terminals well inland on Long Island. When this great project is completed, it will have cost, for real estate, structures, and equipment, $160 000 000.

Besides the tunnels and station, the project comprises numerous large freight yards in New Jersey and on Long Island, with car-float terminals at Greenville, on the New Jersey shore, and at Bay Ridge,

PLAN AND PROFILE OF PENNSYLVANIA TUNNEL AND TERMINAL RAILROA

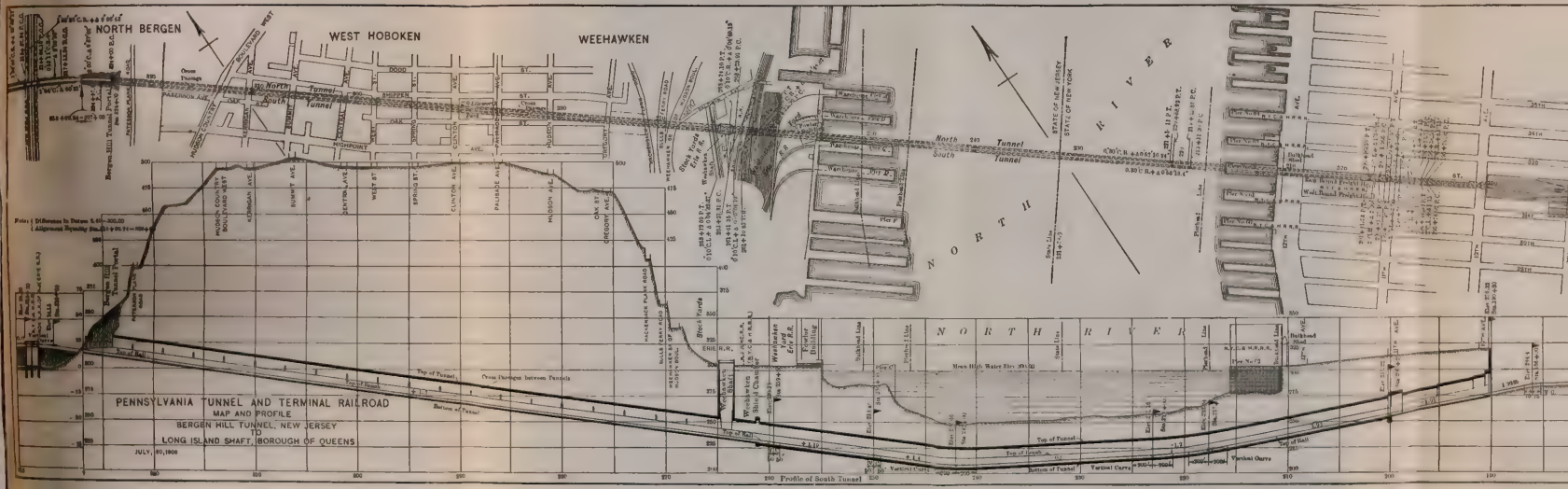

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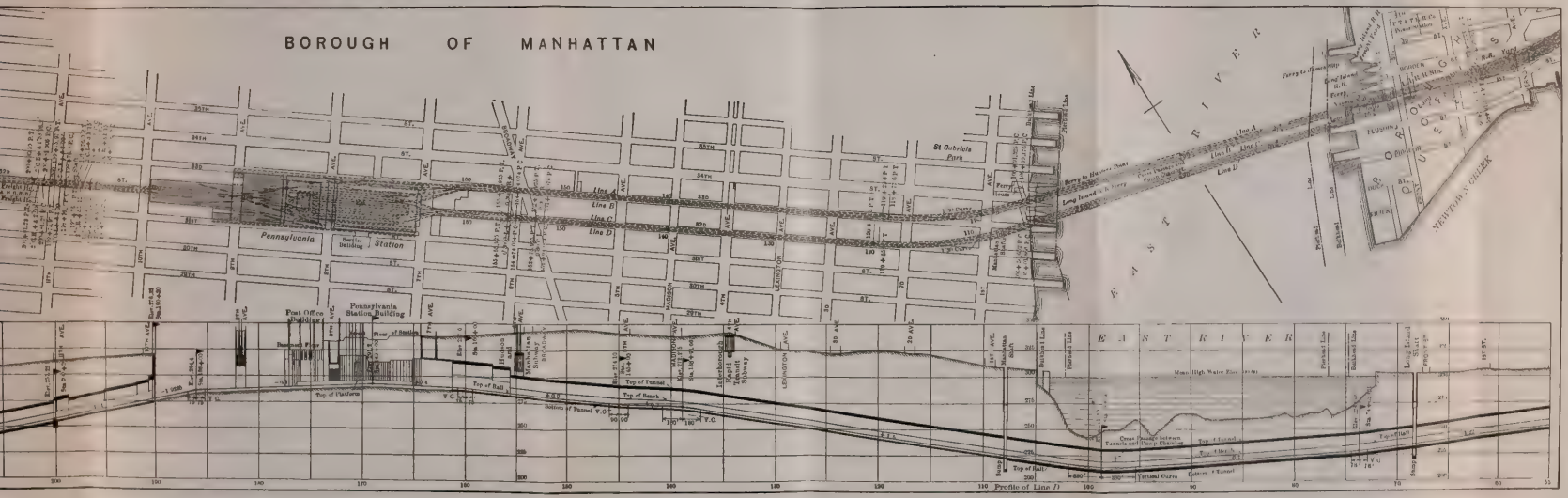
PLATE LXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVI, No. 1274.
FLINN ON
ENGINEERING ACTIVITIES
OF NEW YORK CITY.

D TERMINAL RAILROAD FROM NORTH BERGEN TO LONG ISLAND CITY.



in Brooklyn; extensive revision of the railroads, both in New Jersey and Long Island, involving the elimination of scores of street grade crossings; and the New York Connecting Railroad, 12 miles long. This latter road, now under construction, makes physical connection between the Pennsylvania and Long Island Railroads and the New York, New Haven and Hartford Railroad, thus completing an all-rail route for passengers and merchandise from New England and Eastern Canada to the West, South, and Southwest. To carry the Con-

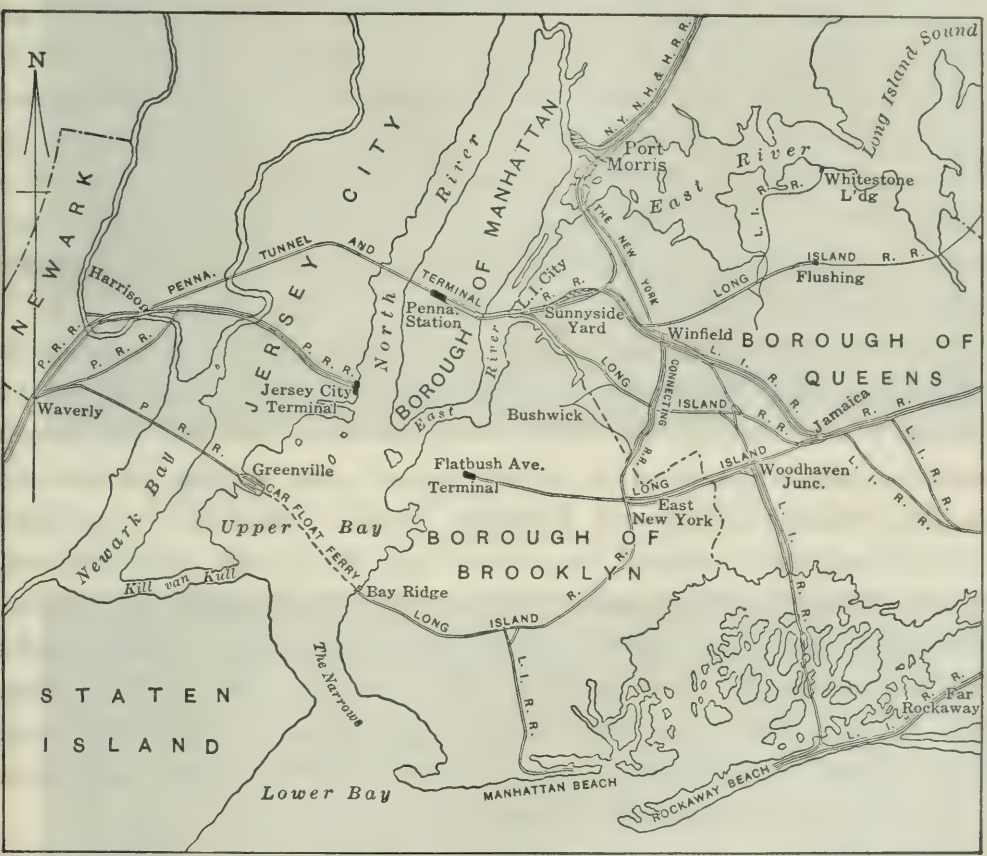


FIG. 22.—MAP OF PENNSYLVANIA, LONG ISLAND, AND NEW YORK, NEW HAVEN AND HARTFORD RAILROADS WITHIN METROPOLITAN DISTRICT.

necting Railroad from Long Island to the main land, a succession of high viaducts and bridges over three estuaries, together 3 miles long, and for four tracks, is being built, crossing the East River at Hell Gate with an arch of 1 000 ft. span and 300 ft. rise, towers 250 ft. high, and a clearance of 140 ft. for vessels. This will be the longest arch span in the world.

It is claimed that Hell Gate Bridge, 15 840 ft. long, will be the longest and largest steel viaduct in the world. Tay Bridge in Scot-

land is 10 780 ft., and the Ohio River Bridge, at Cairo, Ill., 10 560 ft. The Hell Gate arch alone is estimated to cost $6 000 000 of the $30 000 000 total for the Connecting Railroad. Into the bridge structures will be erected 80 000 tons of structural steel, 400 000 cu. yd. of Portland cement concrete reinforced with 4 000 tons of steel, besides large quantities of granite. Some of the steel members will be 10 ft. in diameter and weigh 180 tons each, requiring special heavy cars for their transportation. It has been proportioned for very heavy live loads. Besides the great arch, the bridge will comprise a lift bridge 300 ft. long over Bronx Kill, several long viaducts, and a 1 000-ft. bridge over Little Hell Gate. The construction of the foundations and masonry is now in progress, and the steel-work is being fabricated in the shops. Some of the foundations for the bridge piers will reach a depth of 140 ft. below low water. It is expected that the bridge and other portions of the Connecting Railroad will be ready for traffic by the middle of 1915.

The tunnels beneath the North, or Hudson, River were driven with compressed air, at pressures ranging from 15 to 40 lb. per sq. in., by shields forced through the silt forming the river bottom. A French engineer, who was taken into these tunnels during driving, commented on this concrete instance of American "push." Only about one-third of the material displaced was taken into the shields and removed. Across Manhattan, excavation was largely in rock; near the station, Ninth Avenue and its three elevated and two surface railway tracks had to be supported, for a length of 375 ft., across a trench 60 ft. deep. Beneath the East River, the tunnels were driven by compressed air, with shields, through silt, quicksand, sand, boulders, gravel, and rock. This was the most difficult section.

For driving these tunnels from New Jersey to Long Island seven shafts were sunk, six of which required caissons. Operations were also carried on from portals in New Jersey and on Long Island, and from the great open excavation for the station building and its yard. One shaft was located at each North River margin, one near the middle of Manhattan Island, and two at each shore of the East River. They were large enough to permit the introduction of the tunneling shields. One shaft on the Manhattan shore of the East River was constructed so as to give permanent access to the tunnels. The depth of the deepest shaft was 125 ft., and the greatest horizontal dimensions were 100 by

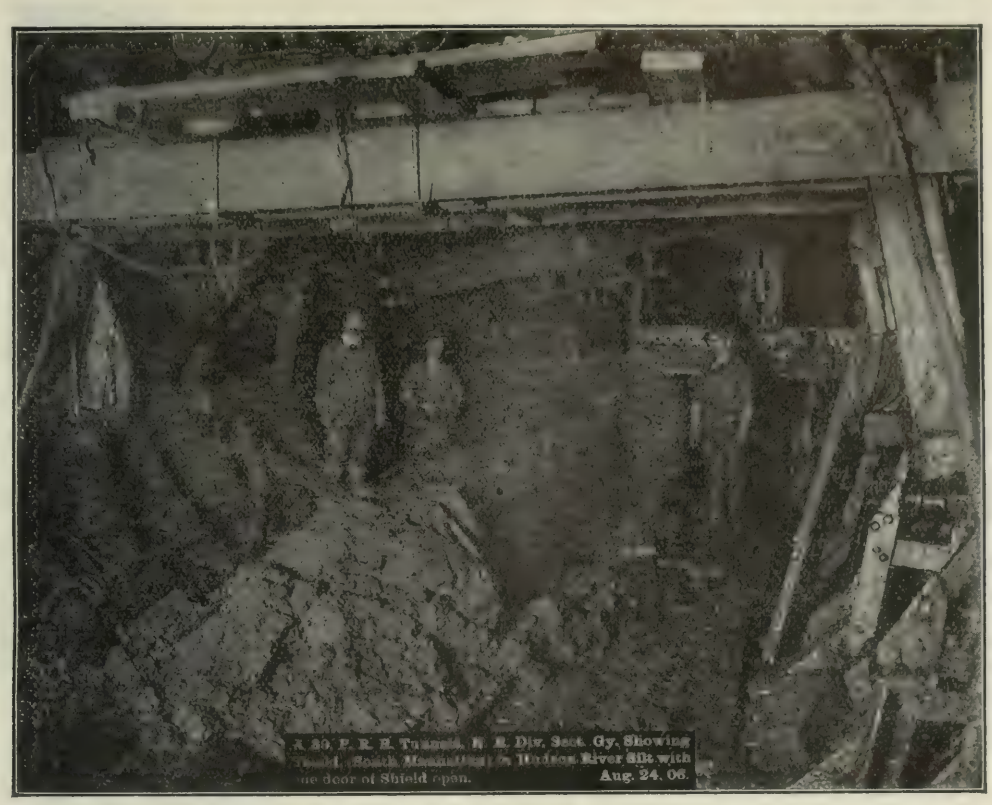


FIG. 23.—PENNSYLVANIA RAILROAD, NORTH RIVER TUNNEL, SHOWING PARTIAL ADMISSION OF SILT THROUGH DOOR WHILE SHIELD WAS BEING PUSHED FORWARD, DISPLACING AND COMPACTING THE REMAINDER OF THE SILT.

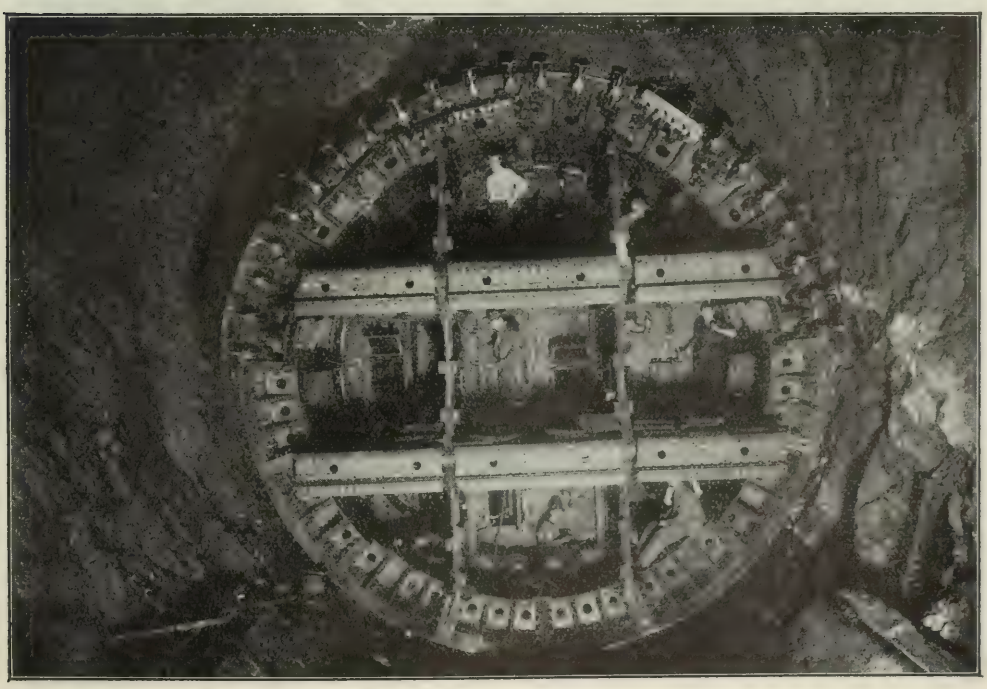


FIG. 24.—PENNSYLVANIA RAILROAD, EAST RIVER TUNNEL, INTERIOR OF SHIELD.

1693.

FIG. 25.—PENNSYLVANIA RAILROAD APPROACH TO STATION IN NEW YORK CITY: NINTH AVENUE, WITH ELEVATED RAILWAY AND SURFACE CARS, ON SUPPORTS ACROSS THE EXCAVATION.

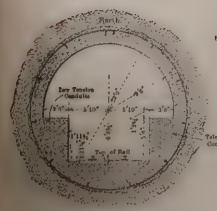
FIG. 26.—PENNSYLVANIA RAILROAD STATION, NEW YORK CITY: INTERIOR OF CONCOURSE, SHOWING STEEL AND GLASS ROOF.

1694

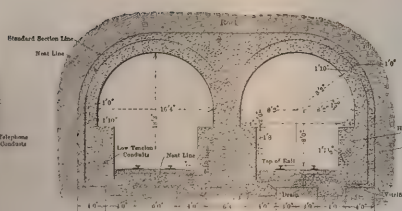
1694A.

PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD.

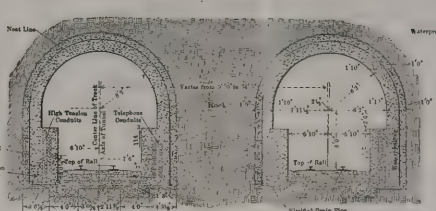
TYPICAL TUNNEL SECTIONS



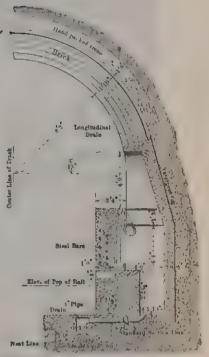
TUBE TUNNEL



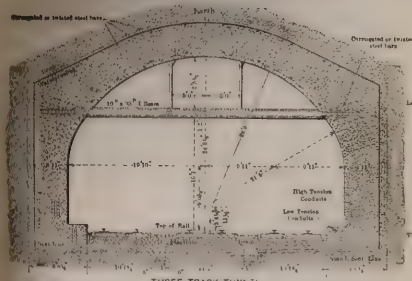
TWIN TUNNEL WITH CONCRETE-DIVIDING WALL



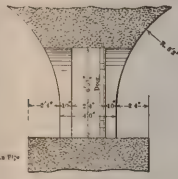
TWIN TUNNEL WITH ROCK-DIVIDING WALL



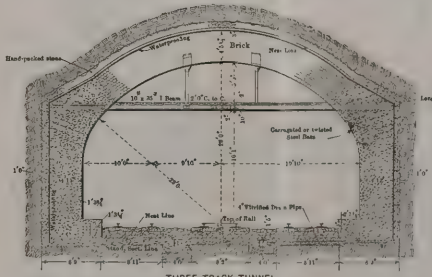
SPLICING CHAMBER
FOR HIGH TENSION CABLES



THREE TRACK TUNNEL
CONCRETE ROOF



CROSS PASSAGE



THREE TRACK TUNNEL
BRICK ROOF

REFUGE NICHE

154 ft. (Weehawken shaft at its top, New Jersey); the smallest shaft was on Manhattan, 22 by 32 ft. and 65 ft. deep. The East River shore shafts are circular, 18 ft. in diameter.

Over the North River tunnels the maximum depth of water is 53 ft., and the tunnel invert, or bottom, is 97 ft. below mean high water. The steepest track grade is 1.93%, and this grade is 1 780 ft. long. These tunnels are 37 ft. apart, center to center. Beneath the East River there are two 1.5% grades, the longer being 4 050 ft. Ventilating fans are installed at the bottoms of the North River shafts, in the building at the Hackensack portal, and in the buildings above the East River shafts. There are also drainage pumps in these shafts. Ventilation, however, is effected largely by the piston action of the moving trains, causing frequent displacements of the air in the tunnels.

Throughout these tunnels, the signaling is three-position upper-quadrant, and, as a whole, represents one of the largest single installations of the kind yet undertaken. The electro-pneumatic system, with interlocking machines and other equipment, and controlling apparatus of the latest and most improved design, is used throughout.

Light signals are used under the Pennsylvania Station and Post Office Building, throughout the Pennsylvania Station Yard, in the North and East River Tunnels, and under the platform shelters at Manhattan Transfer. The use of such a signal in the open—exposed to daylight—is particularly novel, but, as conditions made it imperative, an arrangement of lights and lenses was developed, after a considerable number of experiments, that gives satisfactory color indications under the worst conditions, namely, sunlight directly over the lenses. The change in color, or the giving of the different indications, is effected by the opening and closing of relays, the lenses being colored and placed so as to give the night aspects of the semaphore signals.

Alarm boxes are located in all tunnels at intervals of about 800 ft. These boxes are equipped with two levers; one painted blue and marked "Power", and the other painted red and marked "Fire." If it is desired to cut off the power from the third-rail, the blue lever is pulled, which sends in two rounds of the box number to the sub-station feeding the third-rail, and trips the circuit breakers in the sub-station and switching station. The alarm is also given in the offices of the train director, train and locomotive despatcher, power director, and the

interlocking station controlling the movement of trains in the section in which the trouble occurs. If the red lever is pulled, the power is cut off the third-rail, and four rounds of the alarm are sent in, as above.

The Pennsylvania Station Yard is 3 500 ft. long, from Tenth to Sixth Avenue, and 510 ft. wide, from 31st to 33d Street. Its area is 28 acres, and, to make way for it, 500 buildings were removed, and 3 000 000 cu. yd. of rock were excavated. Concrete retaining walls, aggregating 7 800 lin. ft., were required. The storage capacity of this yard is 222 cars. Beneath the yard and station, 65 ft. below street level, there are 5 200 lin. ft. of trucking subways and pipe galleries.

One of the principal features of the station is the separation of incoming and outgoing passengers. Incoming passengers pass from the train platforms by way of short stairways or elevators to the exit concourse, and thence to the street exits. Outgoing passengers enter the main concourse, and pass to platforms by way of train gates and stairways. The following are some of the principal dimensions and data of this station:

Area of building, 7.74 acres;
Length, 784 ft.; width, 430 ft.;
Building supported on 650 steel columns;
Greatest weight on one column, 1 658 tons;
Weight of building steel, 27 000 tons;
Average height above street, 70 ft.;
Maximum height above street, 154 ft.;
Wire glass used in roof construction, $\frac{3}{8}$ in. thick, 220 tons;
Gross aggregate of concrete used in station, 962 833 cu. ft.;
Water-proofing used, 1 797 964 sq. ft.;
Exterior of building, Milford pink granite, 500 000 cu. ft.;
Brick, 15 000 000;
Number of electric lights in station, in terms of 16-c.p. lamps, 30 000;
Arcade, 46 ft. wide, length, 216 ft.;
Dining-room, 56 ft. wide, length, 112 ft.;
Lunch-room, 56 ft. wide, length 112 ft.;
Main waiting-room, length, 277 ft.; width, 108 ft.; height 150 ft.;
Two sub-waiting rooms, 56 by 100 ft.;
Engagement room, 18 by 28 ft.;
Hospital, 21 ft. 8 in. by 16 ft. 6 in.;
Nursery, 27 ft. by 46 ft. 3 in.;
Two private waiting-rooms for funeral parties, one 16 by 18 ft. 6 in., one 16 ft. 8 in. by 21 ft. 8 in., one equipped with elevator to train platform;

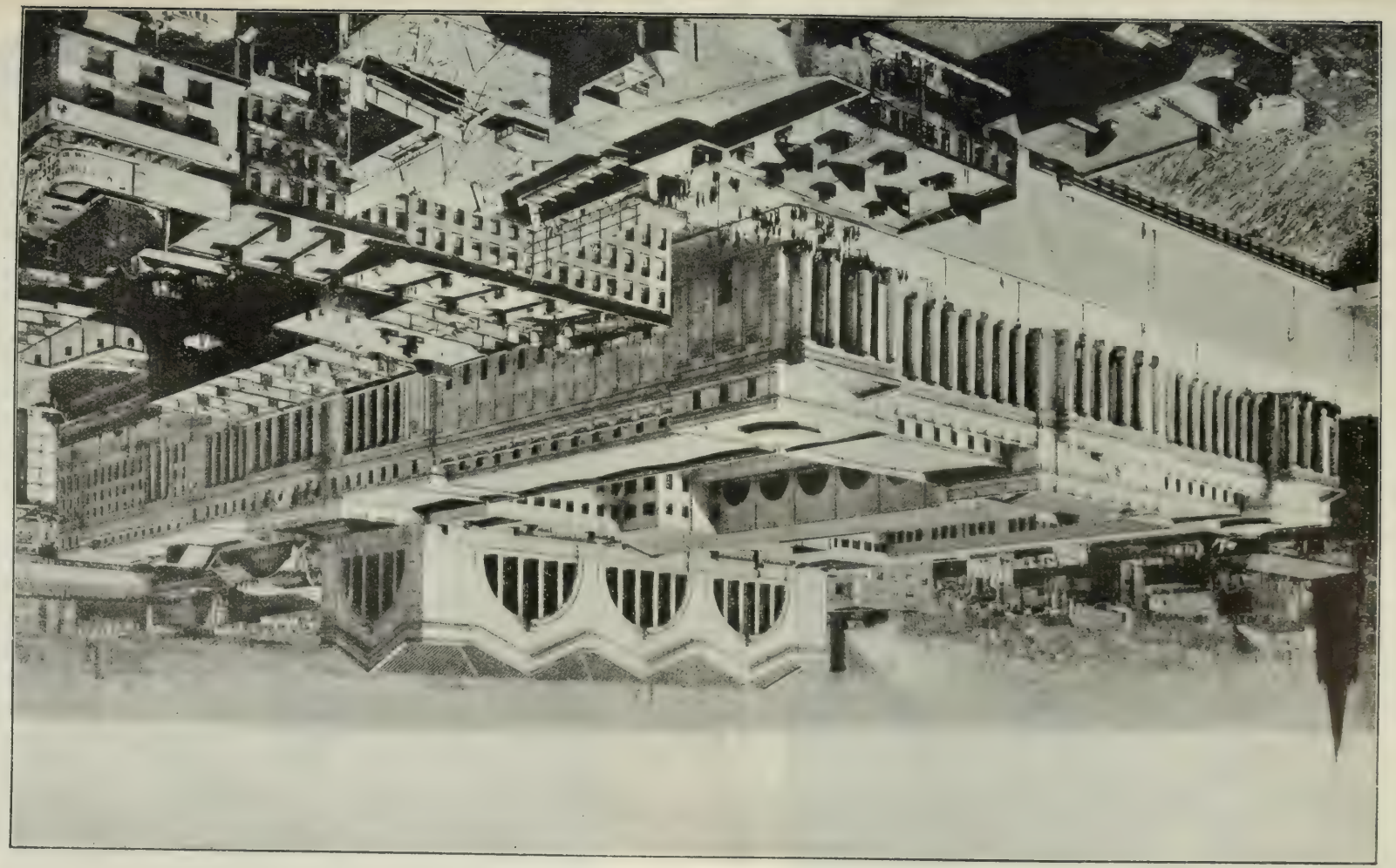


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FIG. 28.—PENNSYLVANIA TERMINAL: GENERAL VIEW OF EXCAVATION, LOOKING WEST. NINTH AVENUE, WITH ITS ELEVATED RAILWAY, IS SUPPORTED ON TRESTLE; EIGHTH AVENUE IS INTERRUPTED; SEVENTH AVENUE, ON WHICH THE STATION NOW FRONTS, IS SUPPORTED PARTLY ON ROCK. HUDSON RIVER IN BACKGROUND.

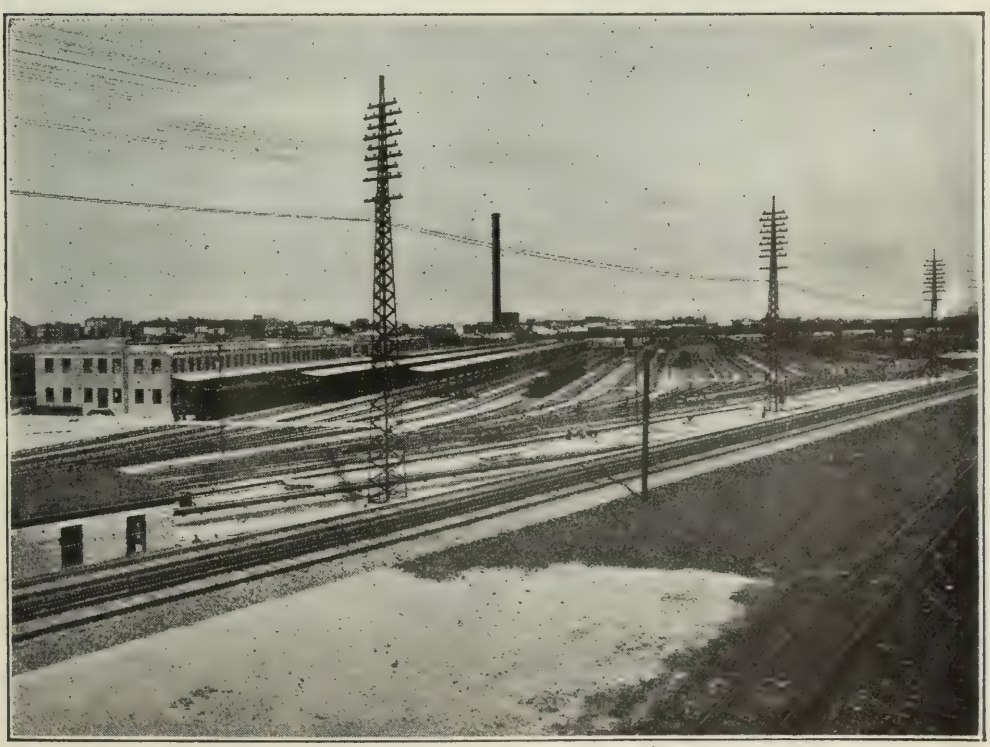


FIG. 29.—PENNSYLVANIA RAILROAD'S SUNNYSIDE YARD, LONG ISLAND. AT THE LEFT IS THE STOREHOUSE FOR REVICTUALING DINING CARS, WHICH THEN PASS AROUND THE LOOP TRACK IN THE BACKGROUND.

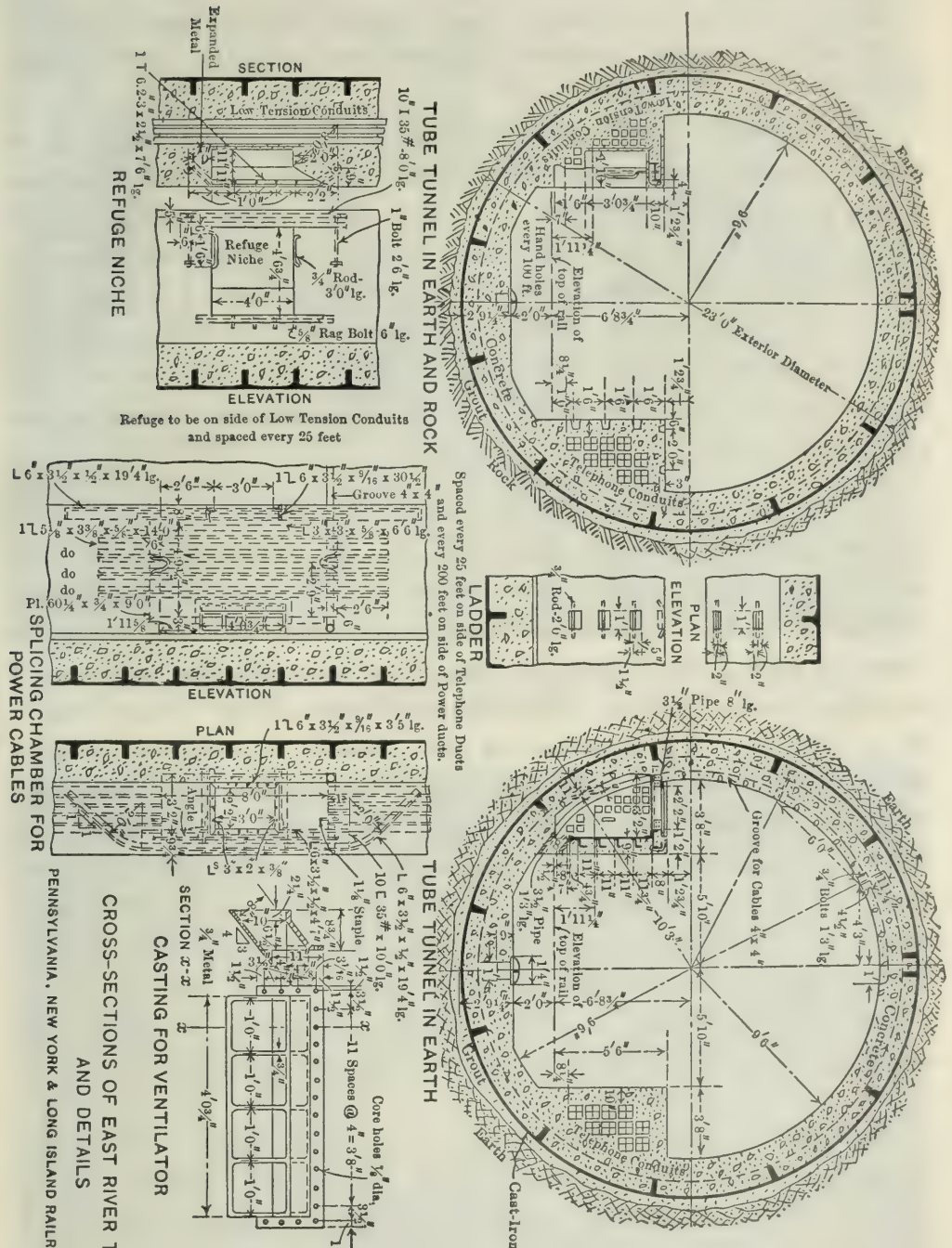


Fig. 30.

CROSS-SECTIONS OF EAST RIVER TUNNELS AND DETAILS

PENNSYLVANIA, NEW YORK & LONG ISLAND RAILROAD COMPANY

Concourse, length, 500 ft.; width, 124 ft.;
Train gates, 36;
Exit concourse, 18 ft. above tracks; width 60 ft.; length 1 088 ft.;
Exits, 34th Street, stairway and escalator, capacity 12 000 people per hour.

The sub-river tunnels have shells 23 ft. in diameter, made up of flanged cast-iron and cast-steel segments, bolted together, weighing 6 tons per lin. ft. All tunnels are lined with concrete and shaped so as to provide foot-paths at the level of the sills of the car windows. Beneath these foot-paths, ducts for wires are embedded in the concrete. The weight of the empty completed tube is 16 tons per lin. ft.; with the maximum train load, it is 23 tons; the weight of silt displaced was 21 tons. The ratio of weight of completed tunnel to displaced silt is 0.74.

Hudson and Manhattan Tunnels.

The Hudson and Manhattan Tunnels now afford to several railroads terminating on the New Jersey shore means for delivering passengers at many points on Manhattan Island, besides serving as a part of New York's street and interurban transportation system and extending out to Newark. The original Hudson River Tunnel, begun about 1874, was long notorious on both sides of the Atlantic because of its physical and financial disasters. The Hudson and Manhattan Railroad Company now has four tubes under the river, a total of 18 miles of tunnel, and a commodious underground terminal containing many spaces for concessionaires, and surmounted by two twenty-two-story office buildings on Church Street, between Cortlandt and Dey Streets, Manhattan, besides numerous subterranean stations. This project, not yet finished, is involving an expenditure of $39 000 000, exclusive of power-house and equipment, and a total expenditure estimated at about $70 000 000.

Beginning in 1902, the final project has progressed steadily. The northerly tunnel of the old pair, from 15th Street, Jersey City, to Morton Street, Manhattan, was holed through on March 11th, 1904, by shield excavation, with compressed air. In the southerly tunnel of this pair a novel modification of shield tunneling methods in soft ground was very successfully tried. The doors in the front of the shield were kept closed at times while the shield was being driven ahead through silt, the soft material thus being forced aside and not taken

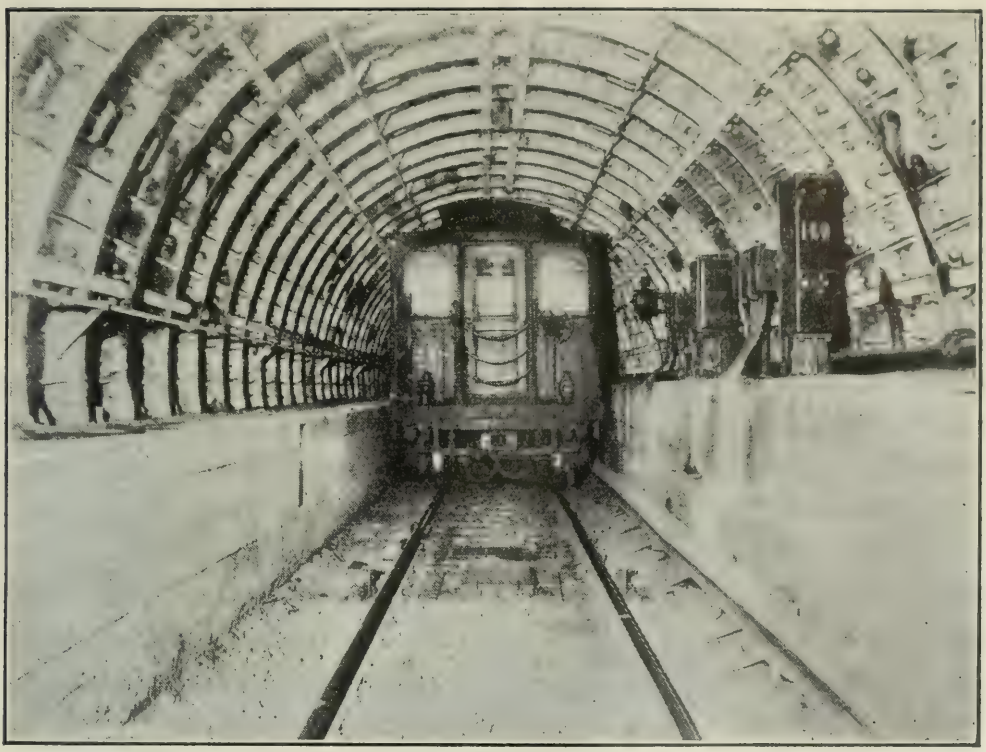


FIG. 31.—"HUDSON TUBES": INTERIOR VIEW, SHOWING CONCRETE SIDE-WALLS AND SIGNAL APPARATUS.

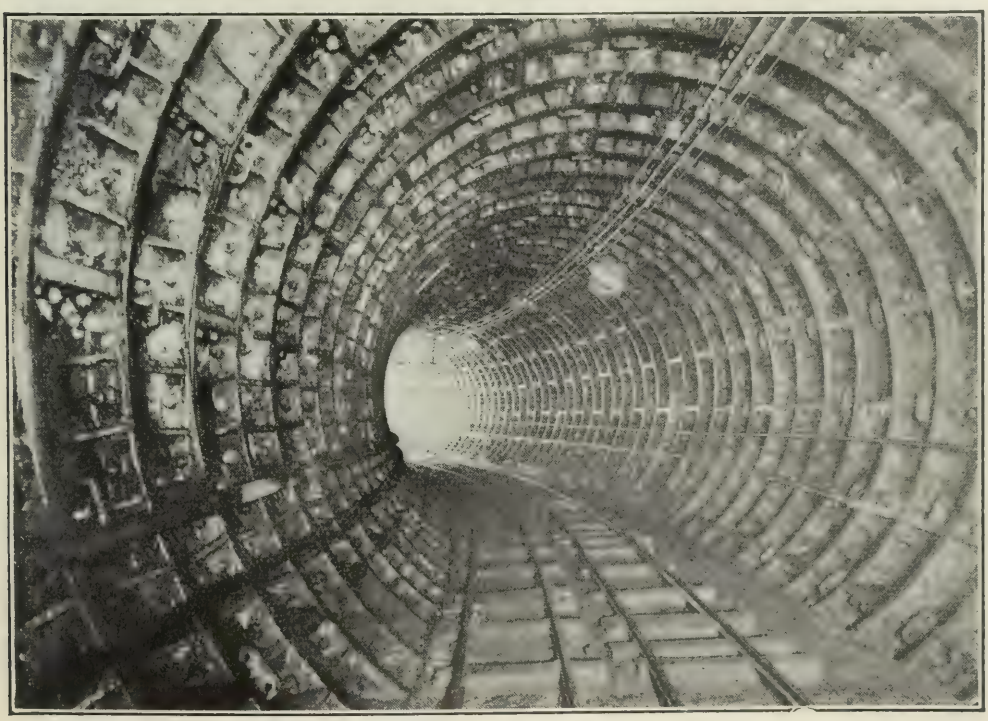


FIG. 32.—"HUDSON TUBES": CURVE IN THE EAST-BOUND TUNNEL AT MORTON AND GREENWICH STREETS, SHOWING CAST-IRON LINING.

inside the tunnel, to be removed at large expense. In this way as much as 62 lin. ft. of progress was made in one day, and in the best week 348 lin. ft. This second tunnel was holed through on September 30th, 1905, its length of 5 100 ft. having been excavated in 334 working days. The outside diameter of the cast-iron lining, or tube, is 16 ft. 7 in., and the inside diameter, 15 ft. 3 in., the flanges being 8 in. deep. The sharpest curves have a radius of 150 ft. This first portion of the system was officially opened on February 25th, 1908.

The southerly pair of tunnels was not in the original project; they extend from the old Pennsylvania Railroad terminal station, in Jersey City, to Cortlandt Street, Manhattan. The southerly tunnel of this pair joined headings on January 27th, 1909. At the deepest place in these tunnels the base of rail is 91.5 ft. below mean tide level. The steepest grade is about 4.5 per cent. The Cortlandt Street tunnels are of the same size as the northerly pair, and were driven largely by the shield displacement method. Here the best day's advance was 72 lin. ft. in three 8-hour shifts; the average speed through silt was 19 ft. per day. Generally the air pressure was about 25 lb. per sq. in., but occasionally a pressure as high as 38 lb. was necessary. All the tunnels passed through some rock near the New York shore. The subway connecting these tunnels in Manhattan is being extended to the Grand Central Station.

Belmont Tunnel.

The Belmont, or Steinway, Tunnel, extending from the Grand Central Station, Manhattan, under the East River to Queens Borough, is a part of the Dual Rapid Transit System, although constructed as a private enterprise and never used. It consists of two 15.5-ft., parallel, single-track, shield-driven, cast-iron-shell, concrete-lined tubes, except where in rock, on shore. The tubes are 28 ft. apart on centers. Under the river, only a portion of the material was admitted through the shields and removed. Most of the excavated earth and large quantities of clay were deposited in deep water over the tunnel to increase the thickness of the roof. At times, the top of a shield was washed bare, and, at other times, boulders were encountered which had to be blasted before the shield could be advanced. The maximum depth of water above the tunnel location was 60 ft. Four shafts were sunk from which to drive the tunnel, two on Manhattan, one on a reef in mid-stream, and the fourth on the Long Island shore. This tunnel is 9 000 ft. long,

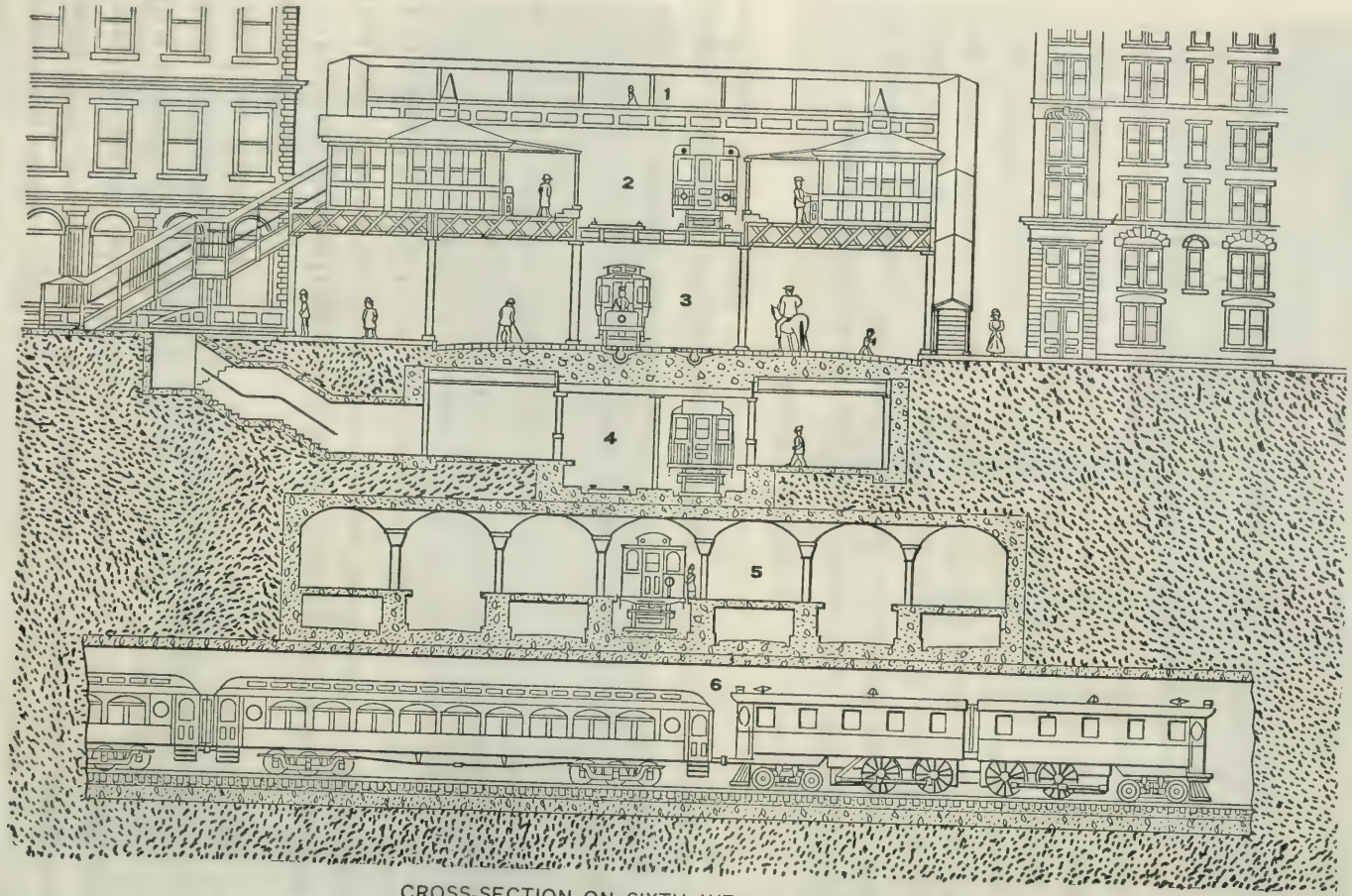
from the Grand Central Station to the Queens terminus, and cost $8 500 000.

Electric Propulsion.

All these great railroad projects, and several others which seem small only by such comparison, have involved the adoption of electricity as the motive power. Indeed, without recent electrical achievements, some of these projects would not have been feasible. Earlier attempts to solve some of these same problems by other means led only to failures.

Following a serious railroad accident in the Park Avenue Tunnel approach to the Grand Central Station, attributed to obscuration of signals by smoke and fog, the State of New York, in 1903, enacted legislation requiring, with other improvements, the substitution of electricity as the tractive power before July 1st, 1908. The "A. C.—D. C." third-rail system was adopted by the New York Central Railroad, with multiple-unit motor cars for suburban service, and 95-ton, 2 200-h.p. electric locomotives for long-haul trains. Cross-connected, duplicate power-houses were built at Port Morris and Yonkers, to generate 3-phase, 25-cycle, alternating current, at 11 000 volts, for transmission by cables, of which a great length is in ducts. Rotary converters, in numerous, conveniently-located sub-stations, change this current to 660-volt direct. Both power-houses are equipped with 625-h.p. water-tube boilers, and 5 000-kw. Curtis turbo-generators. The electric zone extends for 35 miles on the Hudson main line, to Croton Station, and for 25 miles on the Harlem Division, to North White Plains. In July, 1907, the first scheduled trains were run by electric power.

The New York Central Railroad is now having built ten 100-ton electric locomotives of an improved type, bringing its total up to fifty-seven. Although of 15 tons less weight, the new locomotives will be 50% more powerful than the old ones, and have 25% greater speed capacity. Each is rated at 1 400 h.p. continuously, and 5 000 h.p. for short-time maximum. Capable of exerting a pull of 13 500 lb. for one hour at 54 miles per hour, each one of these locomotives is expected to be able to haul a train weighing 1 000 tons, at a speed of 60 miles an hour continuously. There are four trucks of four wheels each under each locomotive, hence they are known as 4-4-4-4 type. A good idea of the appearance of such a tractor may be obtained from Fig. 40.



CROSS-SECTION ON SIXTH AVENUE AT 33d STREET.

1 Foot Passage
2 Manhattan Elevated R.R.
3 Street Surface and Metropolitan St. Ry.
4 Proposed Rapid Transit Subway
5 Hudson and Manhattan R.R. Station
6 Pennsylvania R.R. Tunnel

FIG. 33

On the other hand, the New York, New Haven and Hartford Railroad adopted an alternating-current, overhead-trolley, cradled-catenary suspension system, with 93-ton alternating-current locomotives. These locomotives, however, had to be equipped to operate with direct current over the last 12 miles of track, entering the Grand Central Station. This electric zone extends for 34 miles, to Stamford, Conn., with a power-station at Cos Cob. The current is single-phase, 25-cycle, generated at 4 000 volts, and transmitted at the same voltage to the locomotives.

For the Pennsylvania and Long Island Railroads, a system very similar to that of the New York Central was adopted. Its power-house is in Long Island City, but, for emergencies, connections were arranged at three points: with the power-plants of the Public Service Corporation in New Jersey, the Hudson and Manhattan Railroad Company in Manhattan, and the New York and Queens County Railway in Long Island City.

The electric traction systems of the elevated railways and the subway, first operated in 1902 and 1904, respectively, antedating all the steam railroad electrifications, are also of the "A. C.—D. C." third-rail type, and similar.

Street Railways.

Of street railways on the surface, Greater New York has 635 miles; of elevated railway, 98 miles; and of completed subways, 33 miles; total, 766 miles. It is now possible to travel 18 miles for 5 cents, on one line, without change of cars. These roads together conveyed 1 560 000 000 passengers in 1911, and the traffic increases more rapidly than the facilities. The existing subway, with its equipment, in use since 1904, cost $90 000 000. There are under contract or authorized, 111 additional miles of subway and elevated railway, to contain 312 miles of new track, estimated to cost, with equipment, $325 000 000. Additional third tracks are to be laid on the existing Second, Third, and Ninth Avenue Elevated Railroads. When all these are completed, about 1916, the metropolis will have a system containing parallel lines in zones of heaviest traffic and branches radiating into the various outlying sections affording rapid transportation at low cost. As thus enlarged, the system is known as the Dual System of Rapid Transit, and is expected to have an annual capacity of 3 000 000 000 passengers, exclusive of the surface street railways.

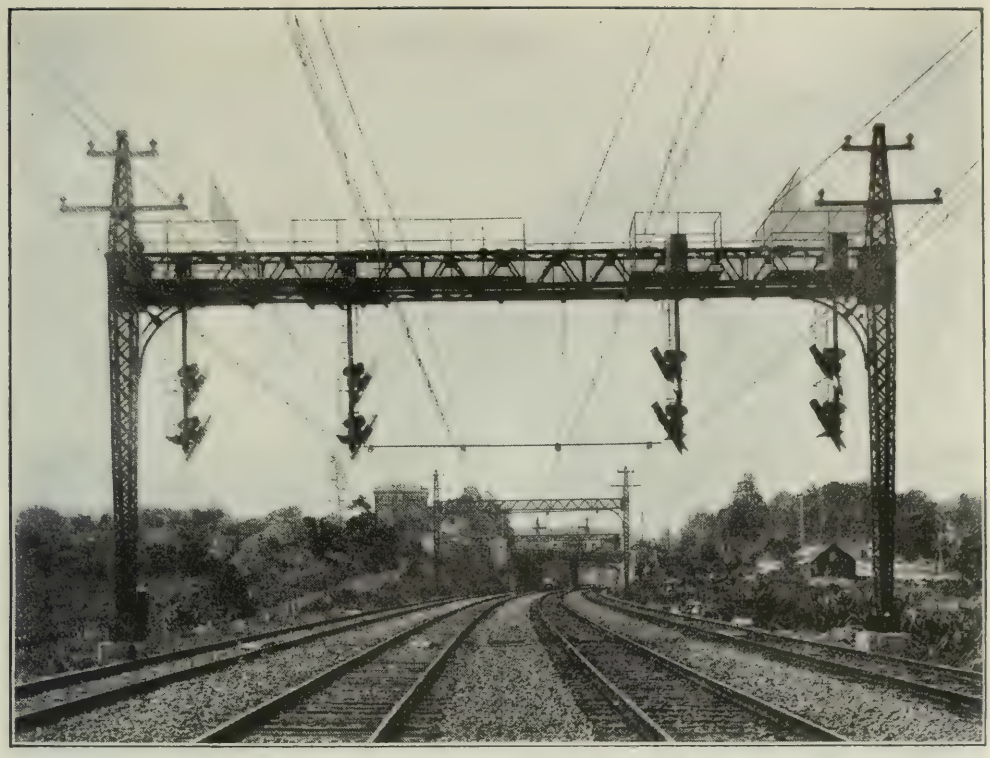


FIG. 34.—NEW YORK, WESTCHESTER AND BOSTON RAILROAD, NEAR MT. VERNON, N. Y., ELECTRICALLY OPERATED. SHOWING SEMAPHORE SIGNAL BRIDGE AND CATENARY SUSPENSION OF TROLLEY WIRES.

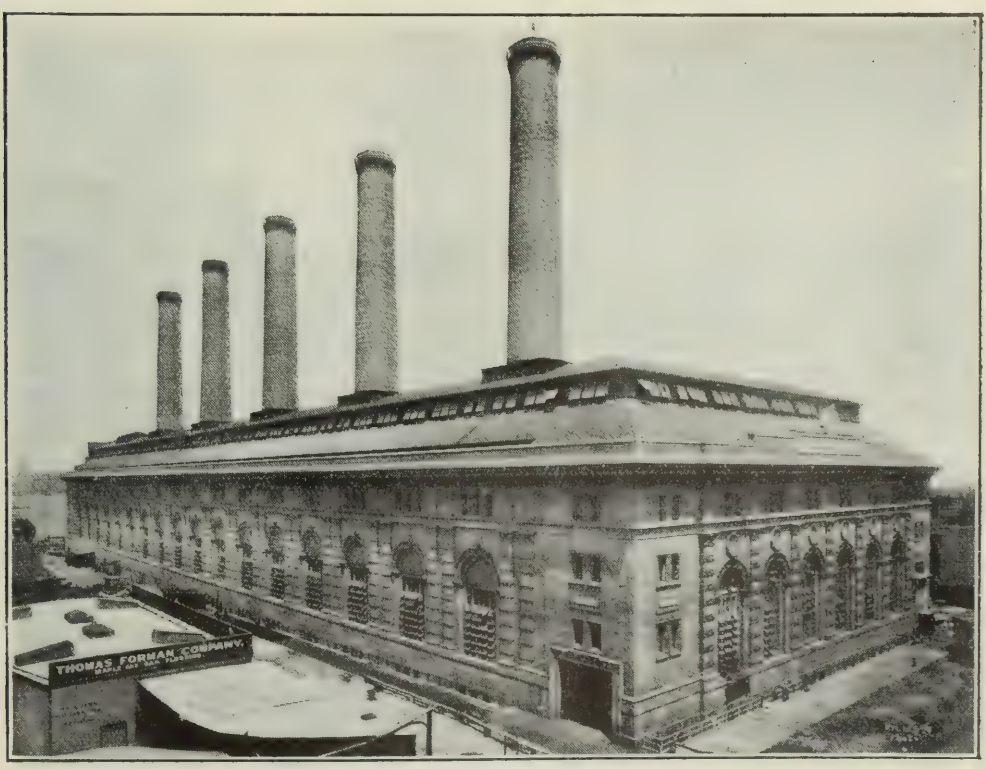


FIG. 35.—INTERBOROUGH RAPID TRANSIT POWER HOUSE, AT FOOT OF WEST 59TH STREET, MANHATTAN.

FIG. 36.—MANHATTAN ELEVATED RAILWAY: CURVE AT 110TH STREET, LOOKING WEST. THE HIGHEST PART OF THE STRUCTURE.

FIG. 37.—BROOKLYN TERMINAL OF BROOKLYN BRIDGE. MANHATTAN BRIDGE (THEN UNDER CONSTRUCTION) IN THE BACKGROUND.

FIG. 38.—CITY HALL LOOP STATION ON FIRST SUBWAY.

FIG. 39.—TYPICAL PROPOSED NEW ELEVATED RAILWAY STATION IN BOROUGH OF QUEENS.

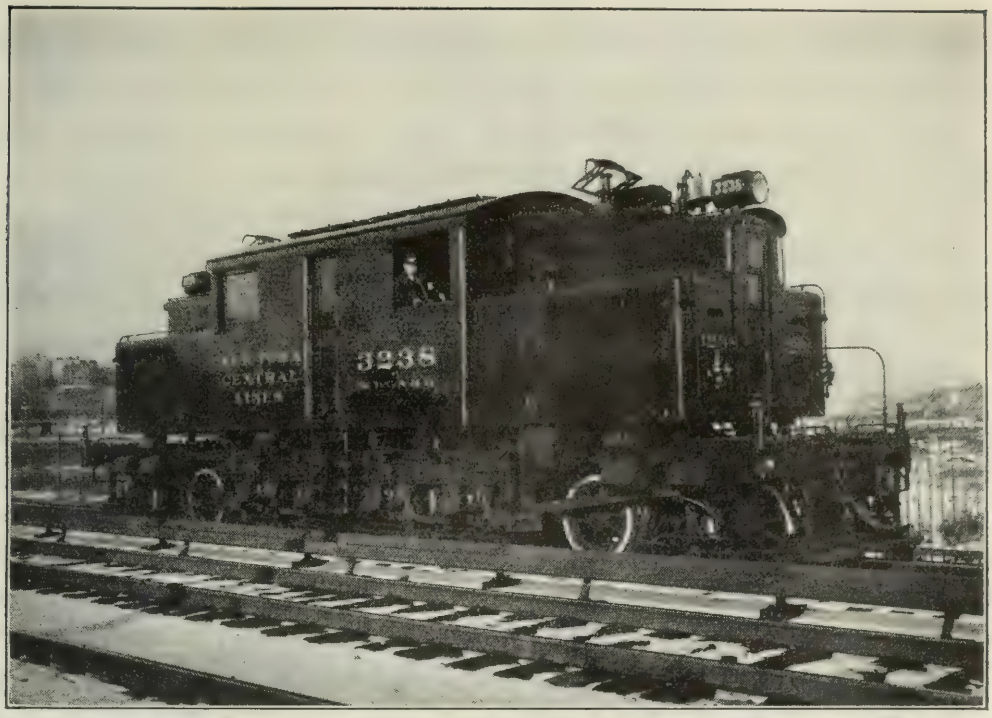


FIG. 40.—NEW 100-TON ELECTRIC LOCOMOTIVE OF THE NEW YORK CENTRAL RAILROAD.

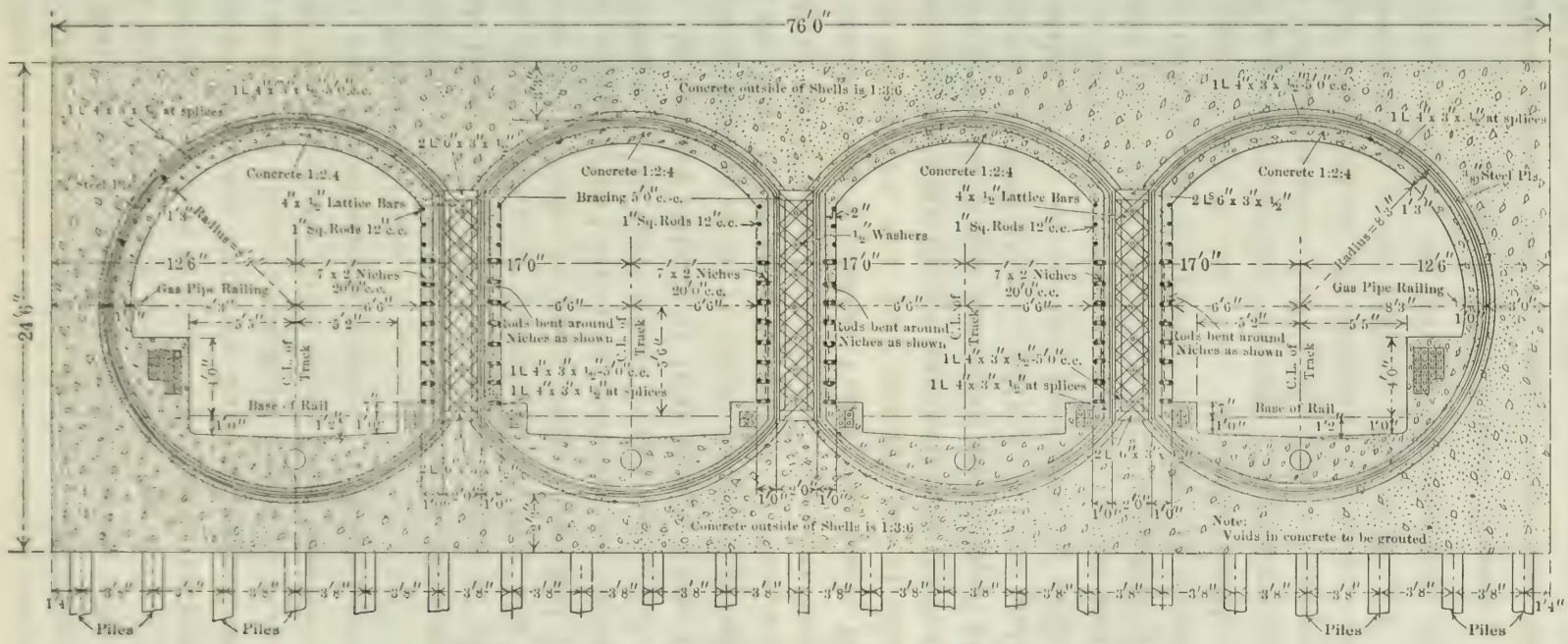
FIG. 41.—STREET RAILWAY CAR OF LATEST TYPE. DOUBLE DECK, WITH STEPLESS SIDE ENTRANCE.

The first elevated roads on Manhattan Island were built about 1870 to 1878, and are still in service, the very earliest being operated by endless chain in 1870. The first dummy engines were put on the road in 1871, and in 1876 forty trains were run daily. The system has been extended from time to time. Until 1902, the lines were all operated with noisy, smoky, dirty, coal-burning locomotives, but about that year, as mentioned above, a third-rail system of electric traction was completed and the fussy little locomotives were relegated to limbo. Brooklyn, also, has an extensive system of elevated railroads, radiating from the Brooklyn Bridge.

Considering the great hosts of passengers, the congested travel in the "rush hours," at the beginning and ending of the business day, the high speeds, and close headway demanded, and the throngs of pedestrians and vehicles in many streets traversed by surface cars, truly the reliability and safety of operation are commendable. Noteworthy features of the City's street transit system are the high reverse curve in the elevated railroad at 110th Street, where the tracks attain a maximum elevation of 60 ft. above the street pavement; the tunnel beneath the East River from Manhattan to Brooklyn; the parabolic steel arch, of 180 ft. span, in Broadway near 129th Street, carrying the subway trains across Manhattan Valley; the 4-mile Fort George rock tunnel of the subway, with deep stations, one of them 180 ft. below the street; the Brooklyn Bridge terminals, and the stations of the subway. The deep stations just mentioned are equipped with large passenger elevators. Several elevated railway stations and the Manhattan Valley station of the subway have been provided with escalators. Excepting a few archaic horse-cars, still in regular service, the first of which were run in 1832, now a curiosity to visitors from other cities, all street railways are electrically operated. In crowded districts the surface cars are run by the "slot," or underground conduit, system; in outlying parts the "trolley" prevails. To produce the necessary current for the various street railway systems, a number of large powerhouses have been erected. As an example, a view is shown of the 59th Street power-station of the subway. The cable as a means of street railway traction had its day between the years 1885 and 1900.

The Dual Rapid Transit System.

Referring again to the new rapid transit lines, the Dual System derives its name from the fact that two companies, the Interborough



CROSS-SECTION OF FOUR-TRACK SUBWAY UNDER HARLEM RIVER.

Note: Special construction will be required to connect Drain Pipes with Sump.
Cross-Connections between all the tracks will be required at two intermediate points.

FIG. 42.

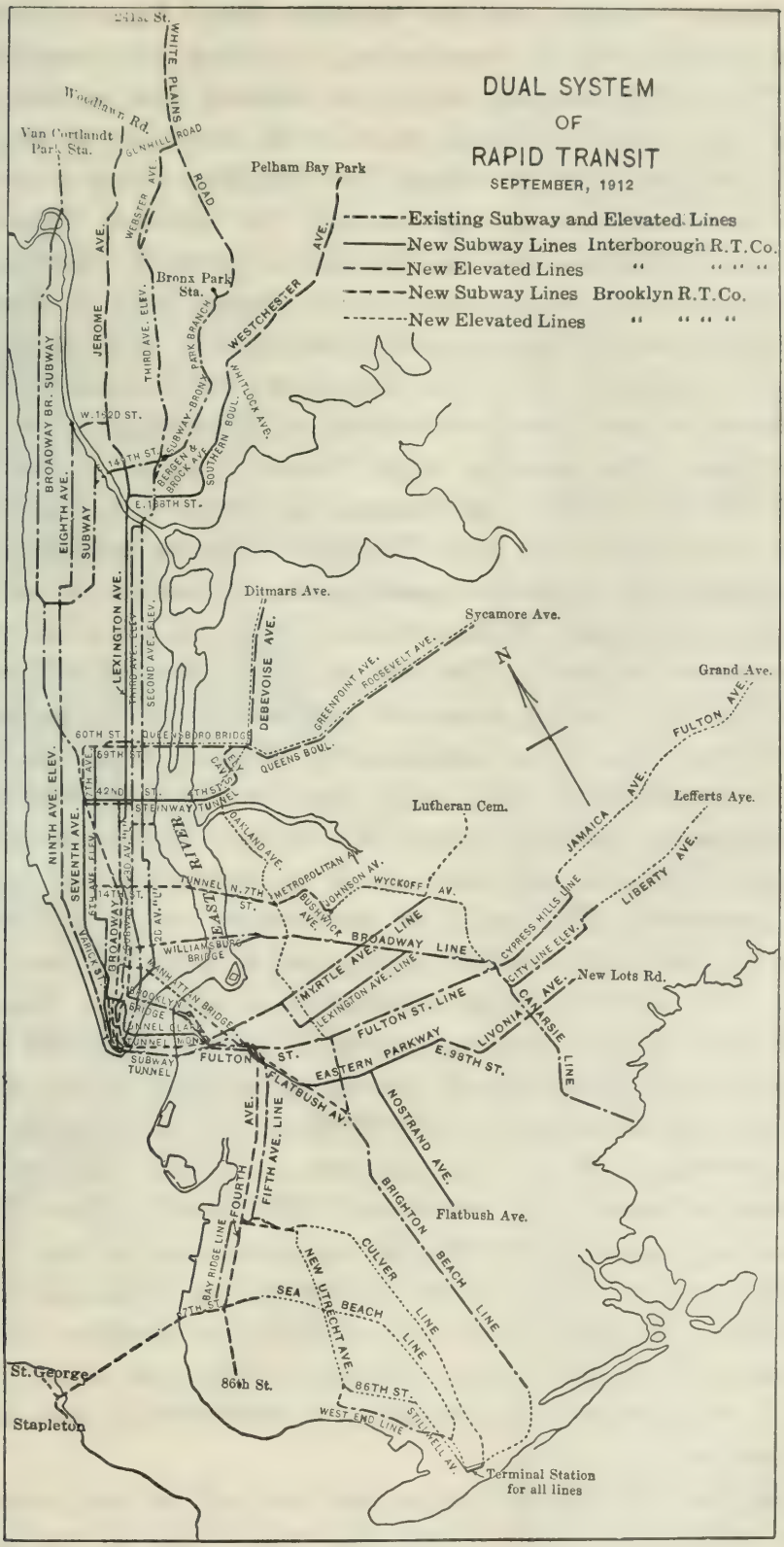
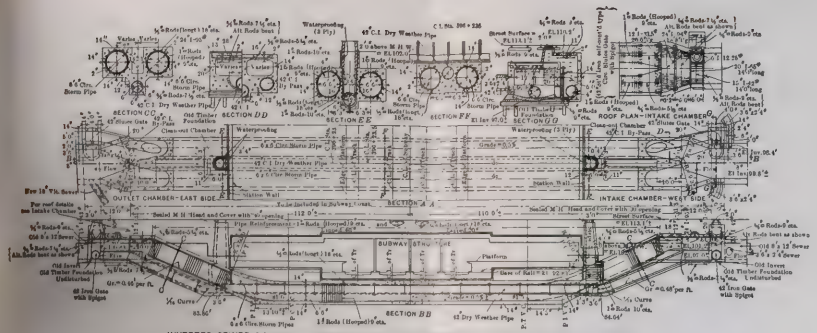

DUAL SYSTEM
OF
RAPID TRANSIT
SEPTEMBER, 1912
Existing Subway and Elevated Lines
New Subway Lines Interborough R.T.Co.
New Elevated Lines " " " "
New Subway Lines Brooklyn R.T.Co.
New Elevated Lines " " " "
241st St.
Woodlawn Rd.
Van Cortlandt Park Sta.
WHITE PLAINS ROAD
DUNHILL ROAD
WEBSTER AVE.
Pelham Bay Park
WESTCHESTER AVE.
Bronx Park Sta.
JEROME AVE.
THIRD AVE. ELEV.
PARK BRANCH
SUBWAY-BRONX
WHITLOCK AVE.
SOUTHERN BOUL.
BROADWAY BR. SUBWAY
EIGHTH AVE. SUBWAY
W. 132D ST.
145TH ST.
BERGEN & BROOK AVE.
E. 138TH ST.
LEXINGTON AVE.
THIRD AVE. ELEV.
SECOND AVE. ELEV.
Ditmars Ave.
DEBEVOISE AVE.
Sycamore Ave.
GREENPOINT AVE.
ROOSEVELT AVE.
QUEENS BOUL.
Grand Ave.
FULTON AVE.
60TH ST.
QUEENSBORO BRIDGE
NINTH AVE. ELEV.
SEVENTH AVE.
42ND ST.
42TH ST.
CATHEDRAL PKY.
STEINWAY TUNNEL
Lutheran Cem.
Lefferts Ave.
JAMAICA AVE.
LIBERTY AVE.
OAKLAND AVE.
EAST RIVER
TUNNEL
N. 7TH ST.
METROPOLITAN AV.
JOHNSON AV.
WYCKOFF AV.
BROADWAY
BUSHWICK AVE.
BROADWAY LINE
CYPRESS HILLS LINE
CITY LINE ELEV.
WILLIAMSBURG BRIDGE
MANHATTAN BRIDGE
BROOKLYN BRIDGE
TUNNEL
MYRTLE AVE. LINE
LEXINGTON AVE. LINE
FULTON ST. LINE
New Lots Rd.
CANARSIE LINE
LIVONIA AVE.
SUBWAY TUNNEL
FULTON ST.
EASTERN PARKWAY
E. 98TH ST.
FLATBUSH AV.
FLATBUSH AVE. LINE
FOURTH AVE.
FIFTH AVE. LINE
NOSTRAND AVE.
BRIGHTON BEACH LINE
Flatbush Ave.
BAY RIDGE LINE
CULVER LINE
NEW UTRECHT AVE.
SEA BEACH LINE
St. George
86th St.
86TH ST.
Stapleton
WEST END LINE
STILLWELL AV.
Terminal Station for all lines

FIG. 43.

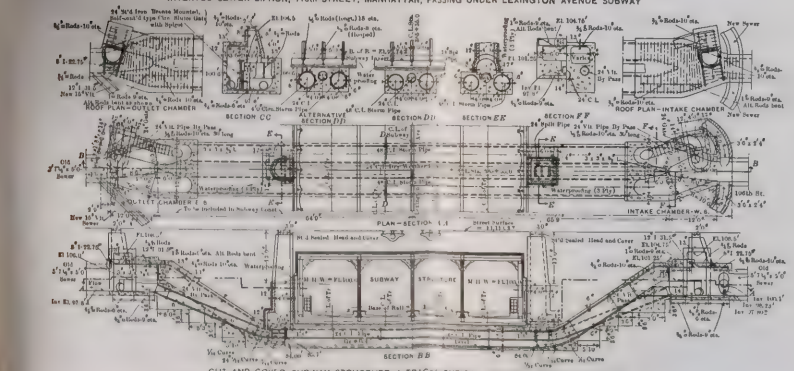
Rapid Transit Company and the Brooklyn Rapid Transit Company, combine with the City in constructing, equipping, and operating the roads. The former company retains the existing lines in Manhattan and The Bronx, and will obtain most of the extensions in these two boroughs, together with the subway into Brooklyn which it now has, certain new connections in that borough, the Steinway Tunnel, extended, and connections over the Queensboro Bridge to Queens. The latter company continues to hold all its elevated lines in Brooklyn, and will get most of the new subway and elevated lines in that borough, and, in addition, most of the lines across the East River Bridges, with important new subway connections on Manhattan Island, south of 59th Street. The Brooklyn Rapid Transit Company is equally interested with the Interborough Company in a number of lines extending into Long Island City, in the Borough of Queens, and will acquire rights to extend the Brooklyn lines through to Jamaica, in the southeasterly part of Queens. Some portions of the lines are to be used by the two companies jointly. The fare is to be 5 cents on each system.

The City is to design, construct, and own the new subways, but agrees to lease them to the companies for 49 years, with definite provisions for recapture in whole or in part, at stipulated times. The companies are to equip and operate the subways. Extensions and improvements of the elevated lines are to be constructed and equipped by the companies. The leases for the elevated road extensions will run for 85 years from completion, but the old franchises are in perpetuity. The City is to share in the profits of operation. The Interborough Rapid Transit Company agrees to expend $56 000 000 for the construction and $21 000 000 for the equipment of new lines, in addition to its total investment of $48 000 000 in the first subway. Thus the Interborough will have an aggregate investment of $125 000 000 in subways, extensions, and equipment. The Brooklyn Rapid Transit Company is to spend $13 000 000 for the construction of new lines to be included in its system, and $21 000 000 for the extensions and improvements of its present lines. In addition, the Brooklyn Company is to devote $26 000 000 to the equipment of the lines which it is to operate. An aggregate new investment of $60 000 000 is thus required of this company.

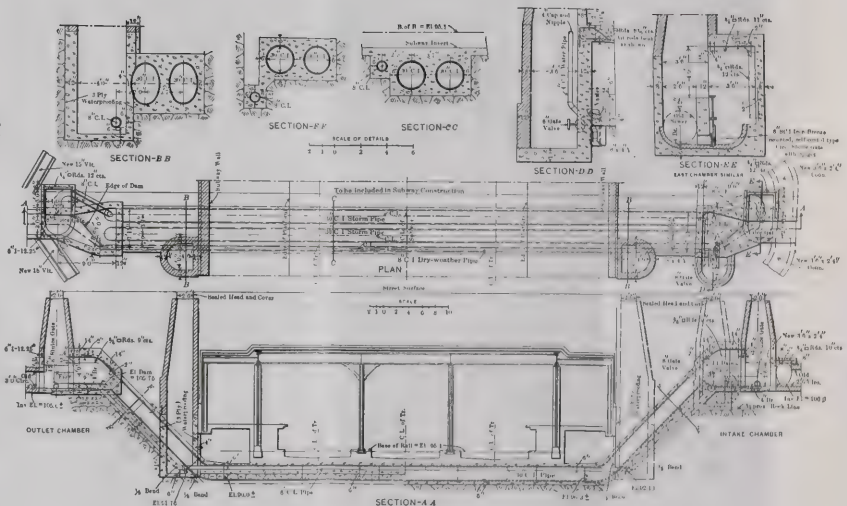
In the new subways, the station platforms will be long enough for ten-car express trains and six-car local trains, and no station plat-



INVERTED SEWER SIPHON, 110th STREET, MANHATTAN, PASSING UNDER LEXINGTON AVENUE SUBWAY



CUT AND COVER SUBWAY STRUCTURE, 4 TRACKS; TYPICAL SECTION BETWEEN STATIONS.
SHOWING INVERTED SEWER SIPHON, IN 106th STREET, MANHATTAN, BENEATH LEXINGTON AVENUE SUBWAY.



CUT AND COVER SUBWAY STRUCTURE
3 TRACKS.
TYPICAL SECTION AT A STATION
WITH INVERTED SEWER SIPHON BENEATH
SOUTHERN BOULEVARD AND 144th STREET,
THE BRONX

forms will be located on curves; this latter feature in the first subway has been found disadvantageous, and even dangerous. Nor will four tracks be placed in one tunnel; experience has indicated that such construction is not conducive to satisfactory ventilation. Partitions will separate the spaces for the pairs of tracks. An endeavor is to be made to lessen the heat in the new subways; in the old subways this becomes oppressive in warm weather. This heat has been attributed to the operation of the great numbers of electric motors and of the brake shoes; the resulting discomfort has been enhanced by the constant and excessive overcrowding of the cars. The new subways are to be larger than the old one. The latter has a height of 12 ft. 10 in. above base of rail and a width of 12 ft. 6 in. for each track. In the new Brooklyn subways, the corresponding dimensions will have a maximum value of 15 ft. and 14 ft., and minimum values of 13 ft. 2 in. and 13 ft. 6 in., respectively. Double-decked subways will be used in narrow avenues and streets.

For the subway structure built in trench, the framework will be of structural steel, and the walls, roof, and partitions of reinforced concrete, the types being similar, in the main, to those of the first subway. Under the Harlem River, the special four-track tube construction shown in Fig. 42 is to be used. In building new subways of the cut-and-cover type, no open trenches are permitted, as was done in the earlier work, where they would interfere with street traffic. Instead, temporary planking, strongly supported, is substituted for the street pavement, and the contractor's operations are carried on beneath. Many difficult problems of detail in the support of elevated and surface railways, and of buildings, and in the relocation of sewers, water and gas pipes, and electric conduits have to be solved. As can be readily appreciated by engineers familiar with operations of this character, the readjustments in the sewerage systems have been particularly troublesome. Solutions of two or three of these problems are given by way of examples and for their inherent interest. The illustrations on Plate LXIII will assist in making the description easier of comprehension. For crossing the Lexington Avenue subway at 106th and 110th Streets, two inverted siphons have been constructed during 1912. By the introduction of these two siphons, it has been possible to keep the subway as near the street surface as ordinary sub-surface pipes permit, saving 10 ft. in depth of excavation through wet material, and about $800 000

in the cost of construction. Incidentally, considerable reduction in the height of stairways is secured to the 110th Street subway station, which is an important consideration, as it affects beneficially the traveling public for all time.

These siphons take the flow from two trunk sewers in their respective locations. The 106th Street sewer has a section 5 ft. 7½ in. high and 5 ft. 0 in. wide, and serves an area of 109 acres, having a run-off of 240 cu. ft. per sec. at times of excessive storms, and a maximum dry-weather, or sanitary, discharge of 9.8 cu. ft. per sec. The 110th Street sewer has a section 8 ft. high and 12 ft. wide, and drains an area of 327 acres, having a storm-water run-off of 356 cu. ft. per sec., and a dry-weather discharge of 29.4 cu. ft. per sec. The following tabulation gives in detail the sizes and lengths of these siphons:

Location. (Manhattan.)	Storm-water pipe.	Dry-weather pipe.	Horizontal length, in feet.	Depression of pressure pipes from grade line, in feet.	Difference of invert elevation at chambers, in feet.
Lexington Ave. and 106th St....	Two 48-in.	One 24-in.	159	13.8	0.15 ±
Lexington Ave. and 110th St....	Two 78-in.	One 42-in.	256	15.5	0.26 ±

These inverted siphons have been provided with the most improved permanent features which have been developed, as follows: The sanitary flow is confined to the small pipe during dry weather, at all stages of tide. The changes of section at the chambers, and all bends, are easy and gradual. The clean-out manholes are located at the cast-iron bends of the dry-weather pipe. Removable split pipes are bolted at the bottom of the clean-out manholes. These provisions make it possible, if necessary, to rod the small pipe between the clean-out manholes and oblique legs between the chambers and the clean-out manholes. By-pass pipes, with sluice-gates, have been added at each chamber, to be used at times when it is necessary to clean the small pipe. With temporary dams erected just outside of the inlet and outlet openings of the small pipe, and the sluice-gate opened, the dry-weather flow will be diverted into one of the storm pipes, and it will then be possible to pump out the water from the small pipe, in order to conduct cleaning operations without interference by the flow.

It is thus seen that the arrangement is such as to render obstructions to the flow unlikely; but, should any occur, ample provisions have

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FIG. 44.—NEW SUBWAY CONSTRUCTION IN LOWER BROADWAY. CONTRACTOR'S BRIDGE, WITH DERRICKS AND SHAFTS AT EACH SIDEWALK, GIVING ACCESS TO EXCAVATION BENEATH TEMPORARY PLANK SURFACE OF STREET.

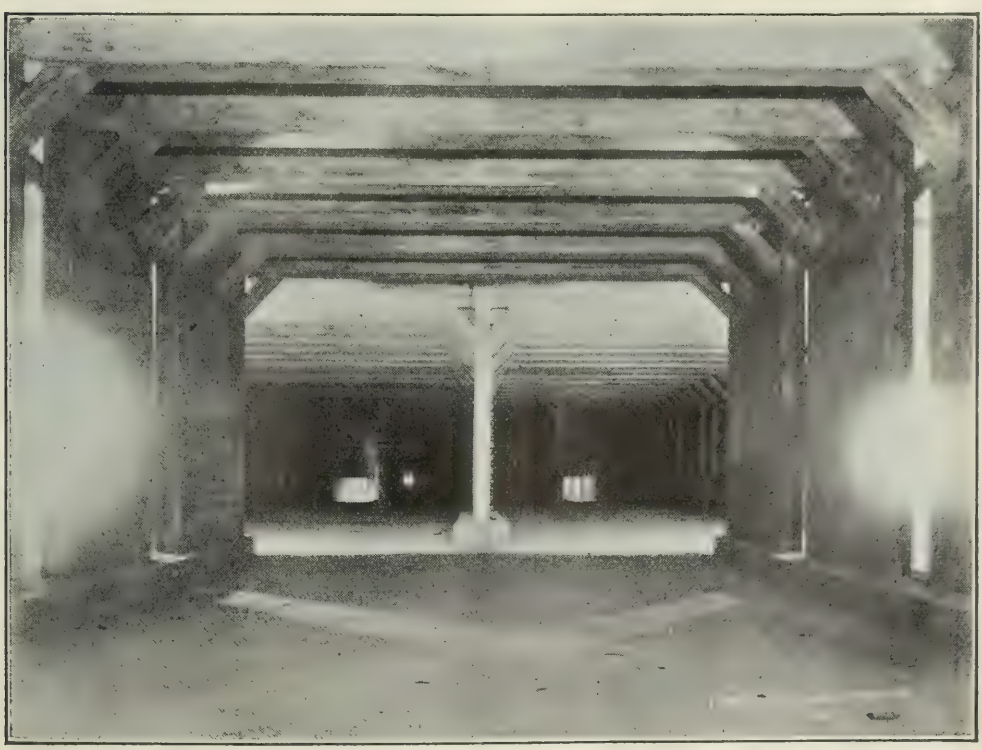

FIG. 45.—CENTRE STREET SUBWAY INTERIOR, READY FOR TRACK CONSTRUCTION.

FIG. 46.—NEW SUBWAY CONSTRUCTION IN LOWER BROADWAY, AT CITY HALL PARK. SHOWS CONTRACTOR'S TEMPORARY BUILDINGS AT LEFT, AND TEMPORARY PLANK SURFACE OF STREET OVER EXCAVATION. POST OFFICE, WITH PARK ROW AND ST. PAUL BUILDINGS IN BACKGROUND.

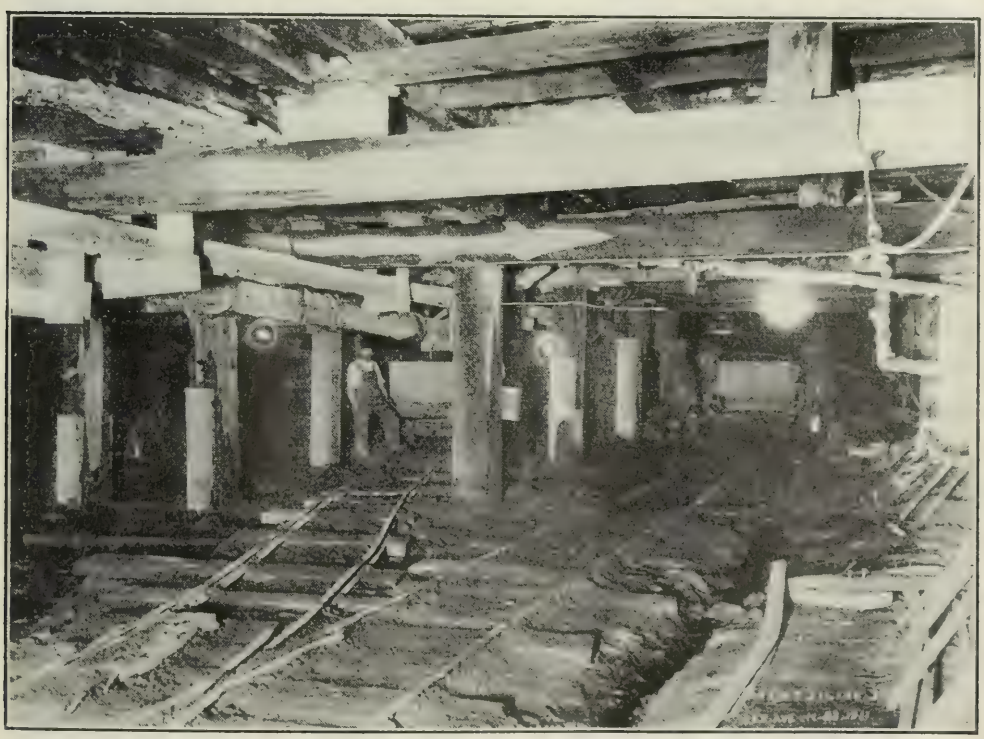

FIG. 47.—SUBWAY EXCAVATION UNDER TEMPORARY PLANK STREET SURFACE, SHOWING SUPPORTS AND METHOD OF SUSPENDING WATER AND GAS PIPES.

been made to remove them. This has been secured satisfactorily without the installation of any motors and pumps, or any movable parts, which would require constant attention and are likely to get out of order.

An inverted siphon designed for Southern Boulevard and 144th Street, The Bronx, in connection with subway construction, is also shown among the illustrations on Plate LXIII.

While this treatment of the sewer problem in connection with subway construction is one which is resorted to in extreme cases only, these siphons are found to work well. They follow substantially the plan and principles of the siphon constructed by the Rapid Transit Commission in 1902 at 149th Street and Railroad Avenue, which works satisfactorily and has never required cleaning. Five inverted siphons have already been built in Brooklyn, in connection with the Fourth Avenue subway work, and the sixth (at Hudson Avenue and Fulton Street) is now under construction. The plans shown on Plate LXIII, which are more or less typical for many cases, are supposed to be the "last word" in sewer siphon design, provision being made for any conditions likely to arise.

For the new elevated railways, the designs adopted are much more pleasing in appearance than the old structure, as indicated by Fig. 39. When all improvements are completed, most of the elevated lines, old and new, will have three tracks, the middle one being for express service "down town" in the morning and "up town" in the late afternoon.

Bridges.

Spanning the East River from Manhattan to Long Island, there are four steel bridges which rank among the greatest in the world. Three are cable suspension bridges, and the most northerly is a cantilever. Brooklyn Bridge, constructed from 1870 to 1883, the oldest and still the most important of them all, has a river span of 1 596 ft., a width of 86 ft., a total length of 6 016 ft., and cost $16 000 000. Nearly 400 000 persons cross it daily. Manhattan Bridge has a river span of 1 470 ft., a total length of 6 855 ft., a width of 123 ft., and cost $14 000 000. Williamsburgh Bridge has the longest span, 1 600 ft., and next to the greatest length, 7 308 ft.; its width is 90 ft., and its cost $14 000 000. Over it pass 250 000 people each day. Queensboro, or Blackwells Island Bridge, the cantilever, has two river spans, of

1 182 and 984 ft., with piers on Blackwells Island. It has the greatest length, 7 449 ft.; it is 90 ft. wide, and cost $13 500 000. Williamsburgh Bridge was opened to traffic in 1903; the other two in 1909. The Brooklyn Bridge towers are about 275 ft. high and the others 325 ft.; the clear height above high tide is 135 ft., for all.

Washington Memorial Bridge spans the Harlem River from the heights of Upper Manhattan to The Bronx, with two 508-ft. steel arches. It is 2 377 ft. long, and was built in 2½ years, at a cost slightly less than $3 000 000.

Pelham Bridge, in The Bronx, is an example of reinforced concrete, with 105-ft. arches. It is 2 000 ft. long, and cost $500 000. Across the narrows in Kensico Reservoir, the Board of Water Supply has built a reinforced concrete bridge 925 ft. long, with five 127-ft. arches having piers of a maximum height of 100 ft.; and at the Ashokan Reservoir a three-hinged reinforced concrete arch of 200 ft. span and 50 ft. rise, with two ribs of I cross-section.

Besides the conspicuous bridges described, several other notable ones have been projected, and there are hundreds, of many types, in existence in New York City, ranging from small wooden highway structures to massive railroad draw-bridges. Most beautiful of all, and possibly the oldest, is High Bridge, 1 450 ft. long, of fifteen semicircular, cut-stone arches, eight 80-ft. and seven 50-ft. spans, on tall masonry piers, supporting the Old Croton Aqueduct over Harlem River, not far from the Washington Bridge. It was built about 1848, at a cost of $1 000 000. Its top is 120 ft. above the river surface. Ten of its piers are on pile foundations.

Bridge projects around New York City cannot be dismissed without at least brief mention of proposed structures across the North River. Up to date, the Hudson has not been bridged south of Poughkeepsie (70 miles north of New York City Hall), where a notable railroad cantilever spans the stream. For years, with intermittent insistence, a bridge of mammoth proportions has been exploited to accommodate railroad and street traffic between lower Manhattan and New Jersey. A clear span of approximately 3 000 ft. being an unalterable condition, the engineering uncertainties and enormous cost have prevented its construction. Successful tunneling beneath the river has rendered it less necessary. However, there is still in contemplation an interstate bridge to afford easier access from upper Manhattan to the high land immediately opposite.

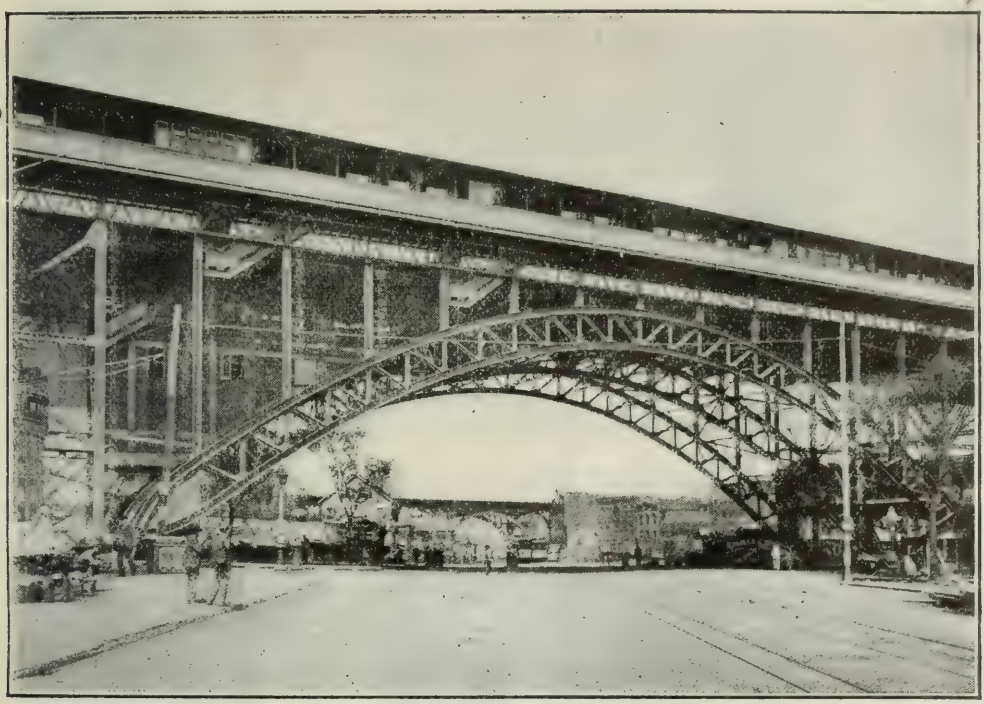

FIG. 48.—PARABOLIC STEEL ARCH, CARRYING BROADWAY SUBWAY TRAINS ACROSS MANHATTAN STREET. STATION ON TOP OF ARCH; ESCALATOR ENTRANCE AT LEFT. STEEL VIADUCT SUPPORTING RIVERSIDE DRIVE ACROSS VALLEY, IN BACKGROUND.

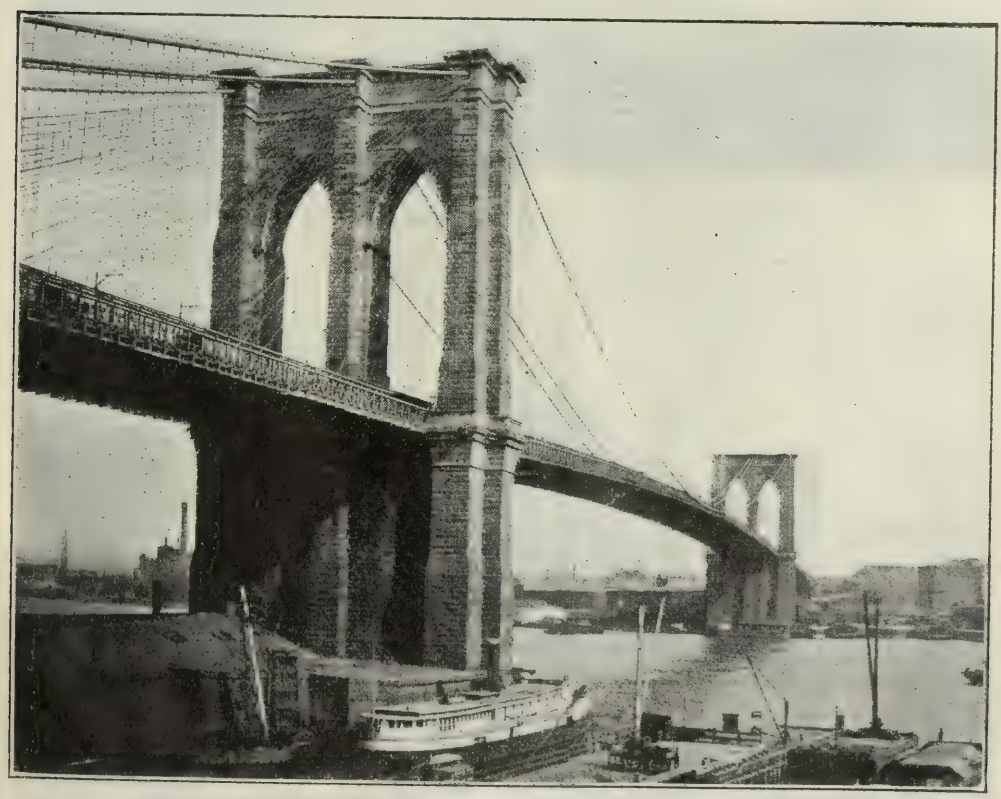

FIG. 49.—BROOKLYN BRIDGE, OVER EAST RIVER.

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FIG. 50.—MANHATTAN BRIDGE, OVER EAST RIVER.

FIG. 51.—WILLIAMSBURGH BRIDGE, OVER EAST RIVER.

FIG. 52.—BLACKWELLS ISLAND, OR QUEENSBORO' BRIDGE, OVER EAST RIVER.

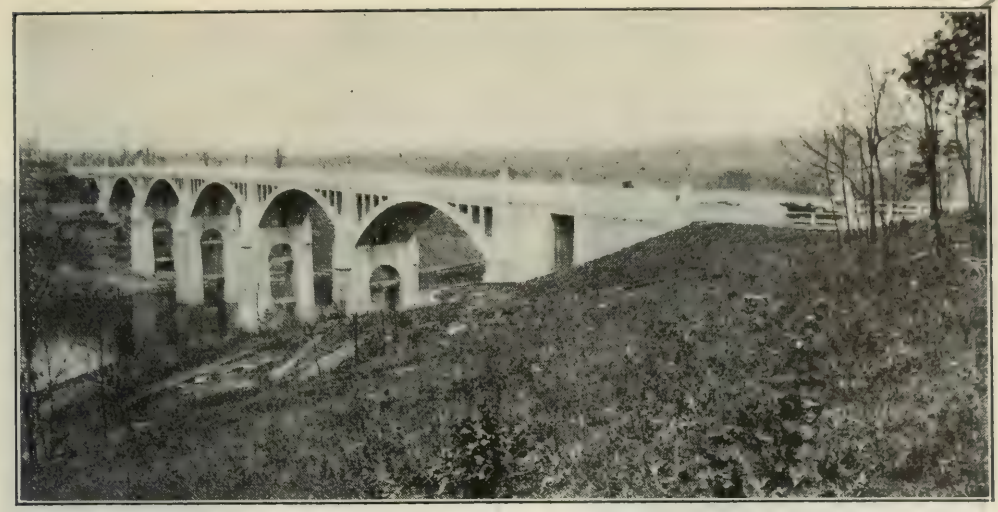

FIG. 53.—RYE OUTLET BRIDGE, AT KENSICO RESERVOIR. REINFORCED CONCRETE; SPANS ABOUT 127 FEET IN THE CLEAR; HIGHEST PIERS 100 FEET. WHEN RESERVOIR IS FILLED, WATER WILL RISE NEARLY TO SPRINGING LINE OF MAIN ARCHES.

FIG. 54.—HIGH BRIDGE, SUPPORTING OLD CROTON AQUEDUCT ACROSS SPEEDWAY AND HARLEM RIVER; WASHINGTON BRIDGE STATION OF N. Y. C. & H. R. R. R. AT RIGHT.

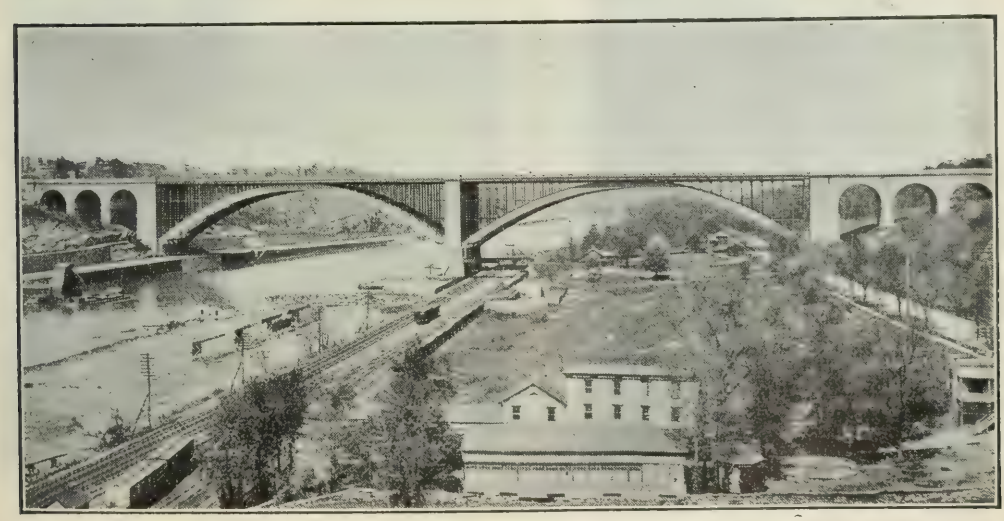

FIG. 55.—WASHINGTON MEMORIAL BRIDGE, CARRYING HIGHWAY OVER NEW YORK CENTRAL RAILROAD AND HARLEM RIVER.

1735.

FIG. 56.—HENRY HUDSON MEMORIAL BRIDGE (PROPOSED), OF REINFORCED CONCRETE.

Formerly it was suggested that this bridge cross at Fort Washington Point (179th Street, Manhattan), but in a recent report the New York and New Jersey Interstate Bridge Commission proposed a bridge at 57th Street and a pair of highway tunnels from Canal Street, Manhattan, to 12th Street, Jersey City. The bridge would be a suspension structure of 2 880 ft. span, center to center of main piers, clear head room of 170 ft. above tide level, and total length of nearly 8 500 ft. Its two main towers would be approximately 600 ft. high and, in addition, would extend 180 ft. below water to rock foundations. The tunnels would be 30 ft. in diameter, driven with shields and lined with concrete inside cast-iron shells; their length would be, roughly, 2½ miles. Each would be for traffic in one direction only. The estimated cost of this great bridge is $42 000 000, and that of the two tunnels $11 000 000. The bridge is intended to carry eight rapid transit and trolley tracks, two four-line roadways, and two sidewalks. Six million vehicles now cross the river by ferries each year, and 158 000 000 passengers by ferries and the existing railway tunnels.

Another great bridge, not yet built, has passed the preliminary stages, and will, doubtless, come into being in the near future. It is the Henry Hudson Memorial Bridge, which will span Spuyten Duyvil Creek and connect the parkway along the Hudson shore of Manhattan Island with a parkway to be laid out in The Bronx, thus forming a continuation of Riverside Drive and Lafayette Boulevard. Planned as a monument to the Dutch navigator's discovery of the river, it is to be of massive granite and reinforced concrete masonry, with a total length of 3 000 ft. Ultimately, this bridge will be a part of the rapid transit system. The most noteworthy of its features is the main, reinforced concrete arch, of 725 ft. span between centers of springing joints, 703 ft. in the clear, with its soffit 180 ft. above the water. The main deck, 215 ft. above the stream, will carry a 50-ft. roadway and two 15-ft. sidewalks; 25 ft. beneath it a second deck will support four rapid transit railway tracks. The main arch rib is designed to be 70 ft. wide, 15 ft. deep at the key and 28 ft. deep at the springing line, with an intrados radius of 485 ft.

Streets.

Greater New York has 3 700 miles of streets, of which 2 000 miles are paved. The principal pavements are of stone, wood, and asphalt blocks, and sheet-asphalt, with much macadam in outlying sections.

STANDARD CROWNS OF PAVEMENTS BOROUGH OF MANHATTAN.

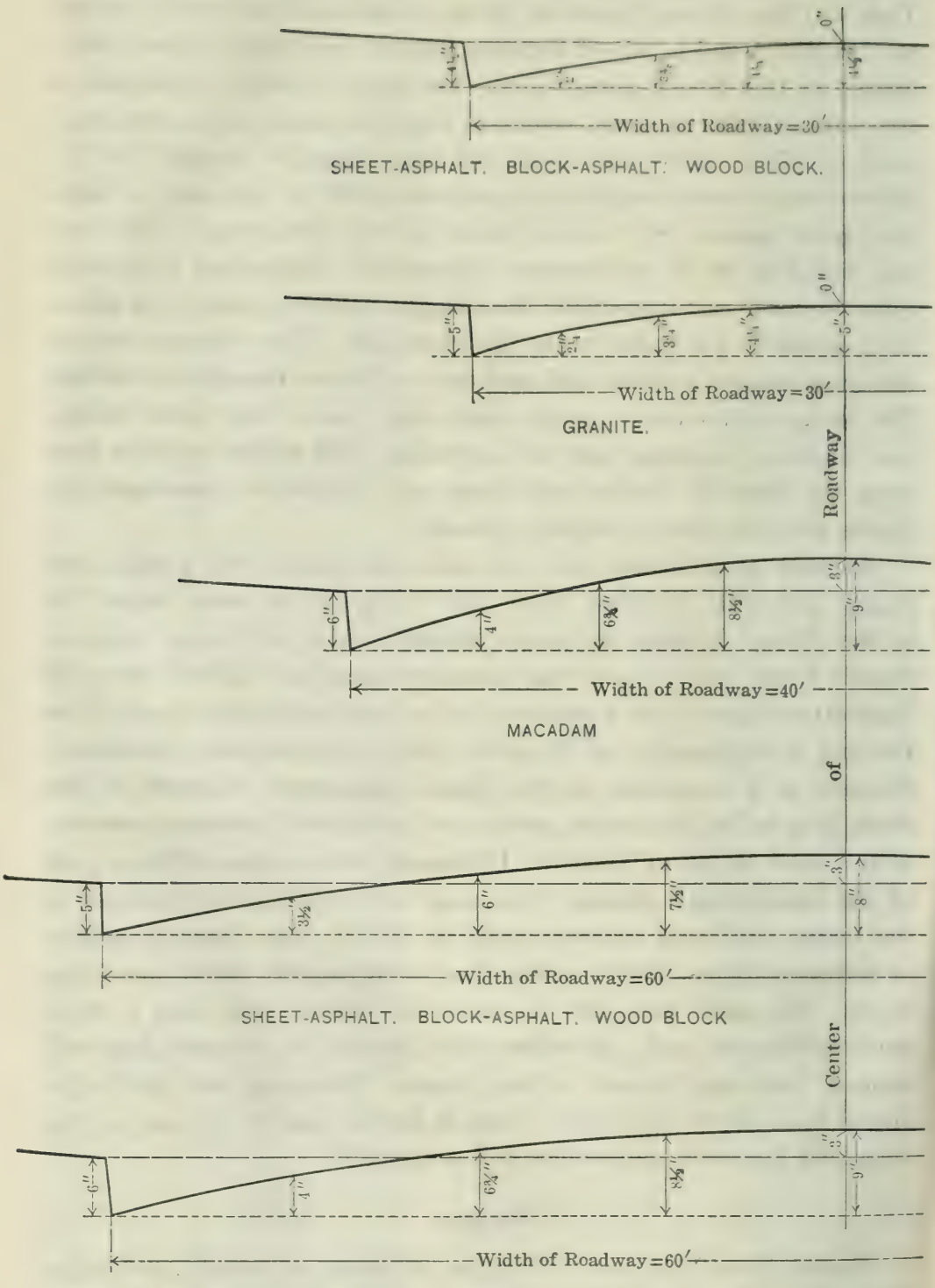

FIG. 58.

As being more or less typical of the street practice of the city, there are shown in Fig. 58 diagrams of the standard cross-sections of several kinds of pavements as constructed in Manhattan Borough. Fig. 59 shows the details where the roadway and sidewalk meet in the gutter and curb, or edgestone. Although this cross-section is drawn for sheet-asphalt pavement, the curb sections for other kinds of pavement are almost identical. The values of the letter dimensions of Fig. 59 are as follows:

STANDARD STREET CURB SECTIONS.

Depth of curb, in inches.	Dimensions, in Inches.		
	A	B	C
5	16	18	17
6	16	18	18
7	14	16	19
8	14	16	20

On many streets traffic is so heavy that the pavements are worn rapidly. The destruction or injury of street surfaces is greatly increased by frequent openings for pipes and other sub-surface structures, which are but indifferently restored.

In all boroughs of Greater New York the streets in the older sections were more or less accidental in arrangement, and but little has been possible in the way of improvement other than an occasional widening or modification of grade; but, generally, the streets of the city are laid out in rectangular systems. In some newer sections street systems planned by real estate companies, with little attention to surrounding neighborhoods, have been perpetuated. Generally, the rectangular systems have uniform spacings between streets in any given section; but in the oldest districts, and in some others where topography has been permitted to control, crooked streets of irregular width and spacing are found, with grades also less skilfully regulated.

CURB SECTION FOR SHEET-ASPHALT PAVEMENT.

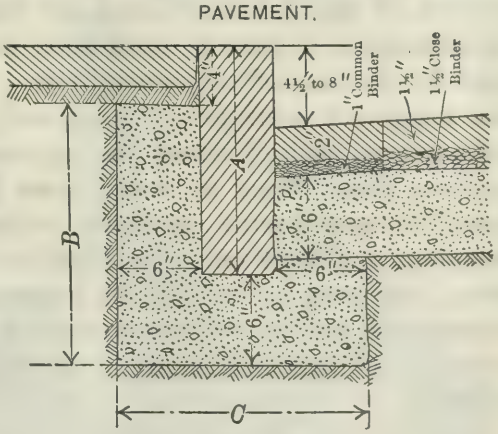

FIG. 59.

Under a rule of the Board of Estimate and Apportionment, no street with a width of less than 60 ft. will be placed upon the City map unless it is less than 1 500 ft. in length, or unless the damage which would result to buildings is greater than the probable benefit resulting from widening. The street widths, therefore, may be said to range from 60 to 100 ft., the former being that usually fixed for the minor streets, while that for the more important arteries ranges from 70 to 100 ft., with an approximate average of 80 ft. An effort has been made to absorb all private developments within the City plan, this course being essential for the reason that the City is without power to assist property owners in closing streets which have been dedicated unless the title can be recaptured by the abutting owners or unless the City is in possession of the fee. In most of the old private streets the fee has been lost, and in all such cases the Board attempts to adjust the old lines by straightening them as necessary and by providing an adequate width. An excellent illustration of this treatment is afforded in a recent map of a large area in the Borough of Queens, concerning which the engineers of the Board recently made a report, from which the following is quoted:

"The map comprises the sections known as Hollis, Holliswood, Bellaire, Queens, St. Albans, Springfield, Laurelton, Rosedale, and a portion of the Jamaica Estates. The Jamaica Estates and Laurelton developments appear to have been based on a systematic and suitable plan for an area of substantial size. With the exception of Holliswood, the remaining sections are composed of a large number of separate developments, each of which has apparently been carried out along independent lines, these in many cases being inconsistent with those followed in neighboring territory, making the problem of comprehensive mapping a difficult one, the solution of which, however, has in general been attempted by laying out highways to follow the former sectional boundaries, and by widening existing thoroughfares and laying out new ones as required to secure a network of traffic arteries of adequate width."

This may be taken as typical of the solution usually attempted, although in many instances the result has not been as satisfactory as in this case.

Manhattan has a comprehensive street scheme which has been followed consistently and rigorously for many years. In accordance therewith, the island has been developed north of the old city, the boundary of which may be drawn at 14th Street, or approximately

FIG. 60.—CITY HALL AND PARK, LOOKING NORTH, BROADWAY AT LEFT.

FIG. 61.—WOOLWORTH BUILDING, POST OFFICE, AND CITY HALL, WITH SURROUNDING "SKY-SCRAPERS," FROM THE MUNICIPAL BUILDING.

1742.

2¾ miles from the Battery. The plan is a gridiron, with avenues, from 100 to 150 ft. wide, parallel to the north-south axis of the island, spaced ⅛ mile apart, and cross streets, at right angles to the avenues, spaced twenty to the mile. A few intermediate avenues have been added in the central part of the borough. Several cross streets are 100 ft. wide, but the standard width of the others is 60 ft., a 30-ft. roadway and two 15-ft. sidewalks. Even where heavy rock cuts have been necessary, not only have the streets been laid out on these lines and built to established, easy grades, but the lots between have been excavated to conform thereto. Broadway, the old colonial post road to Albany, is the only important diagonal street in this gridiron, and only for a part of the way is it diagonal. A street system for practically the whole area now constituting the Borough of Brooklyn was established by a map about forty years ago. That portion of the Borough of The Bronx west of the Bronx River was mapped in 1895.

Many old streets in congested districts are proving inadequate for their traffic, the sidewalks as well as the roadways being too narrow. Illegal encroachments have aggravated these conditions, and finally the City has resorted to drastic remedies. In lower Broadway and adjacent streets, for example, buildings are being shorn of architectural adornments and other parts which project beyond the legal building line, at a cost of hundreds of thousands of dollars. Instead of disfiguring the buildings, as might have been supposed, the work has been done so skilfully in many cases that appearances have been improved. Rarely have the changes been objectionable.

As is evident along many of New York's streets, in some places painfully, little or no control is exercised over the relationship of buildings to their surroundings as to height, architectural style, or color. Such inexcusable and deliberate inharmonies as the City Investing Building, beside the Singer Building, should not again be perpetrated. However, unnecessary ugliness has finally attracted intelligent attention, and definite efforts are being directed toward the creation of more beautiful structures and groups. Several civic centers are definitely being planned. Architecturally, New York has made notable improvement during recent years.

Street Cleaning.

In Queens and Richmond the borough president has charge of the street cleaning, but in the remaining boroughs the Department

of Street Cleaning is charged with this function. Its duties are: (1) Sweeping and cleaning, sprinkling, flushing or washing, and sanding of the streets; (2) Removal of ashes, street sweepings, garbage, light refuse, and rubbish; (3) Removal of snow and ice from leading thoroughfares and from other streets to the extent practicable. In the 124 sq. miles served by the department are 1 900 miles of streets, of which 1 300 are paved. From the 24 000 000 sq. yd. of pavement cleaned, 546 000 tons of sweepings are gathered yearly, by about 3 000 men. Of ashes, more than 2 000 000 tons, of garbage 315 000 tons, and of light refuse and rubbish nearly 190 000 tons were carried off in 1911 by 1 425 collectors; and 904 000 cu. yd. of snow were removed from falls aggregating 18.7 in. Ashes, street sweepings, and similar refuse are used for filling low-lying lands, about 35% going to Riker's Island, where 63 acres have been reclaimed from the tide and 173 additional acres are being filled. Salable wastes, such as paper, tin cans, rags, bottles, rubber, and leather, are picked out during the handling of the rubbish. Offal and garbage, including dead animals, are treated by a digesting process, recovering fats, hides, hair, glue, and fertilizer.

Public Buildings.

Of public and semi-public edifices worthy of remark, New York, considering its youth, has a creditable number. Besides those mentioned in other connections, the City Hall, the Stock Exchange, the Custom House, the Public Library, the Metropolitan Museum of Art, the American Museum of Natural History, Columbia University, New York University, with its Hall of Fame, the College of the City of New York, St. Patrick's Cathedral, Cathedral of St. John the Divine (still in course of construction), St. Luke's Hospital, General Grant's Tomb, and the Brooklyn Institute of Arts and Sciences, are conspicuous in their several localities.

Many of the public school buildings catch the eye of the passer-by. Morris High School, in The Bronx, erected in 1900, is English Gothic in style, and cost $648 000. It accommodates 2 531 boys and girls, and contains two good gymnasiums, physical and chemical laboratories, and one of the best auditoriums in the city schools, capable of seating 1 200 persons. Erasmus Hall, Brooklyn, a group of buildings arranged about a quadrangle, after the English college style, is not yet completed; the buildings in place were erected between 1904 and 1910

1745

FIG. 62.—COLUMBIA UNIVERSITY LIBRARY.

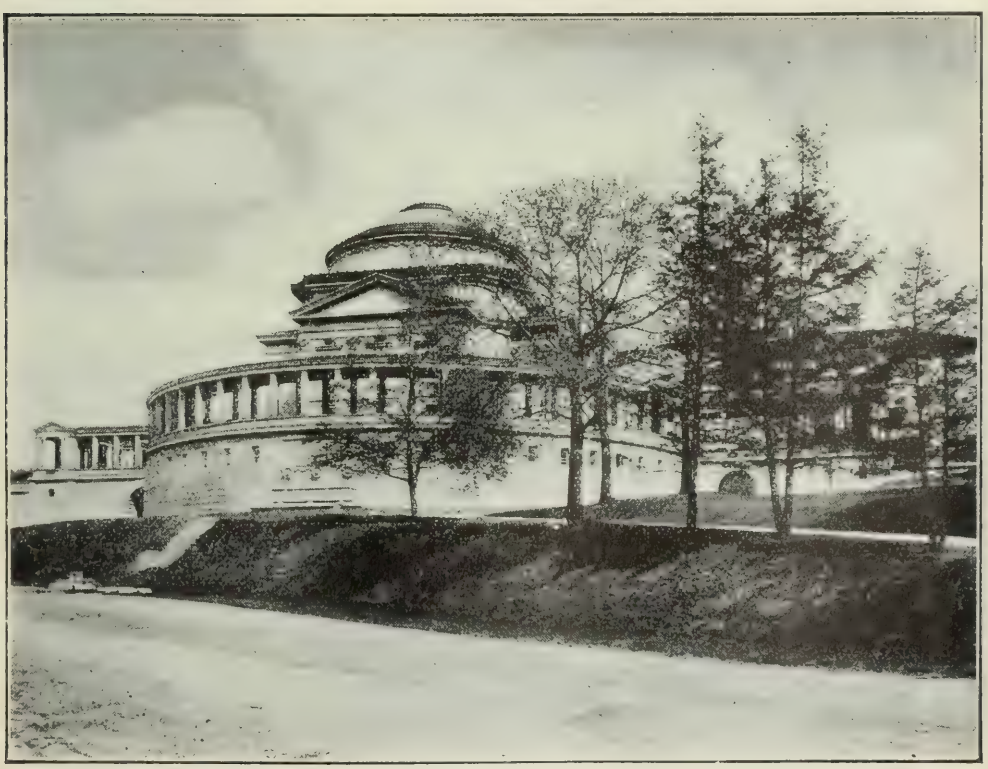

FIG. 63.—HALL OF FAME, NEW YORK UNIVERSITY.

1747

FIG. 64.—COLLEGE OF THE CITY OF NEW YORK.

FIG. 65.—RIVERSIDE PARK AND DRIVE, FROM MILITARY MAST OF BATTLESHIP "KENTUCKY," SHOWING GENERAL GRANT'S TOMB, AT EXTREME LEFT, AND TYPICAL HIGH-CLASS APARTMENT HOUSES OF UPPER MANHATTAN WATER-FRONT ON HUDSON RIVER.

1749

FIG. 66.—EPISCOPAL CATHEDRAL OF ST. JOHN THE DIVINE, ON MORNINGSIDE HEIGHTS, MANHATTAN, AS IT WILL APPEAR WHEN COMPLETED.

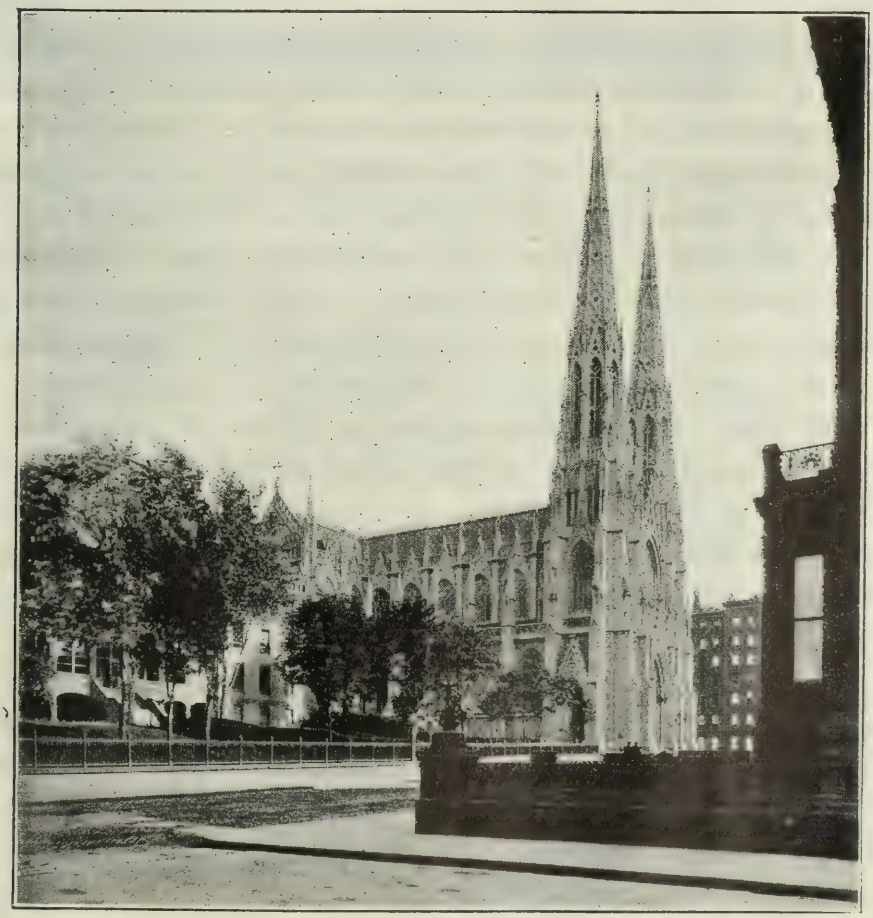

FIG. 67.—ST. PATRICK'S CATHEDRAL, FIFTH AVENUE.

at a cost of $770 000. It has detached buildings for different phases of work, such as physical and chemical laboratories and gymnasiums, and accommodates 2 500 boys and girls. Manual Training High School, Brooklyn, erected 1901-02, cost, including its equipment, $770 000, and accommodates 2 500 boys and girls. It contains machine, wood-working, foundry, and tin shops, with complete equipment, for boys; dressmaking, millinery, cooking, and housekeeping departments, for girls. DeWitt Clinton High School, Manhattan, erected 1903-04, at a cost of $892 000, accommodates 3 500 boys. It contains a gymnasium and full laboratory equipment, also a lunchroom in the top story. Public School No. 62, Manhattan, facing Seward Park, erected 1903-04, cost $660 000; it has a large auditorium, a gymnasium, and a domestic science department, and accommodates 4 250 boys and girls.

Famous all over the country, New York's shops, especially the prominent department stores, deserve passing mention, for their buildings and equipments are largely the products of engineering activities. Of hotels, New York has many which, for convenience and luxury of appointment, are unsurpassed, and of famous restaurants, not a few. The Plaza, one of the most conspicuous among the hostelries, appears incidentally in one of the illustrations. New York, however, can boast no opera house or theatre of great architectural charm with fine setting; but, at a cost of $20 000 000, it is erecting a Municipal Building to provide offices for many city departments for which accommodations are now rented. This huge building is situated near City Hall; Chambers Street pierces the lower part, and there are subway stations in the basements. A few figures will suggest its size: Total floor area, 850 000 sq. ft.; height of tower, 590 ft.; cubic contents of building, about 15 000 000 cu. ft.; cost, 70 cents per cu. ft. It required 37 horses to haul each of the twelve 50-ton girders, over the Chambers Street arch; two 30-ton derricks transferred each girder in mid-air to two other 30-ton derricks to set it in place. There are 40-ton stones on the concourse floor; 30-ton stones in the architrave course on the fourth floor; the column sections, from the first to the fourth story, weigh about 18 tons; the column caps at the fourth story weigh about 16 tons; on the twenty-first story the column sections weigh 10 tons; on the thirty-fifth story some of the stones weigh 10 tons. The foundations are of the pneumatic caisson type, common among the tall

buildings south of 14th Street. Some of the caissons are the deepest ever sunk for a building, being 147 ft. The basements are below tide. Like other modern New York office buildings, the Municipal Building is of "steel-cage" construction, and is faced with granite. It will have thirty-two passenger elevators traveling from the street level to the twenty-fifth story, and special elevators in the tower above. It will have a very complete mechanical and electrical equipment. Every effort is being exerted to render the structure and most of its furnishings unburnable.

"Sky-Scrapers."

Approaching the City from any direction, by day or night, its excessively tall buildings fascinate the beholder, and continued observation only increases the wonderment, as their number, the enormous value, and the great multitudes of persons housed, penetrates one's comprehension. The "sky-scrapers" constitute the most characteristic physical aspect of the City on lower Manhattan Island. Tallest of all is the Woolworth Building (750 ft.), which has recently been completed. Quite as wonderful as this structure are the rapidity and precision with which it has been created and the organization which has done it. Within 18 months from the day the foundations were started, many offices, already rented, were ready for occupancy. Following in height and interest are the Metropolitan Tower, in Madison Square, 700 ft.; the Singer Tower, 612 ft.; City Investing Building, 500 ft.; Park Row Building, 382 ft.; "World," "Tribune," "Times," Bankers Trust, Whitehall, and Hudson Terminal Buildings, with scores more almost equally deserving of mention.

On Manhattan Island there are 1 200 buildings having 10 stories or more; of these, 120 have more than 16 stories, and 15 have more than 25 stories. Of the following numbers of stories there is one building each: 31, 32, 38, 41, 45, and 55; and three buildings have 33 stories. The total number of buildings on the island is estimated at 120 000. For "sky-scrapers" the average height is 13 stories.

Nearly or quite all the great public and business buildings of the City have been created by the collaboration of engineers and architects. By the Civil Engineer many interesting foundation and structural problems have been solved. The hauling of steel members weighing as much as 60 tons apiece has severely tested street pavements and sub-surface structures, and the transportation and erection of such

FIG. 68.—MORRIS HIGH SCHOOL, BOROUGH OF THE BRONX.

FIG. 69.—PUBLIC GRAMMAR SCHOOL NO. 62, BOROUGH OF MANHATTAN.

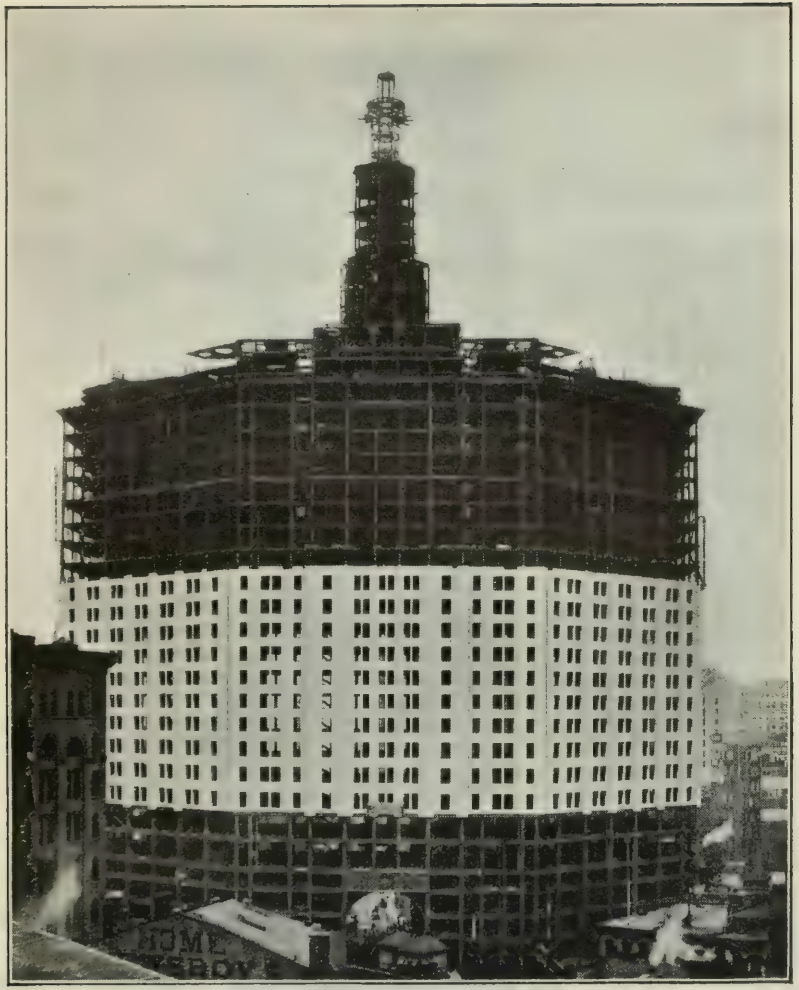

FIG. 70.—MUNICIPAL BUILDING, EAST FAÇADE, ILLUSTRATING A FAMILIAR STAGE IN THE ERECTION OF A STEEL-CAGE SKY-SCRAPER

FIG. 71.—MUNICIPAL BUILDING, WEST FAÇADE; CITY HALL CUPOLA IN FOREGROUND; OLD COURT HOUSES AT LEFT; WITH HALL OF RECORDS BACK OF THEM

Height to which Croton Water rises by gravity
Catskill Water will rise without pumping to this height
Broadway
Home Life Ins. Co's Bldg

FIG. 72.—CONSPICUOUS BUILDINGS IN LOWER MANHATTAN: COUNTY COURT HOUSE (FOREGROUND), CITY HALL AND POST OFFICE (CENTER), ON BACK ROW, LEFT TO RIGHT, "WORLD," "TRIBUNE," "AMERICAN TRACT SOCIETY," "TIMES" (DOWN-TOWN BUILDING), PARK ROW, ST. PAUL, SINGER TOWER, CITY INVESTING, HUDSON TERMINAL (EXTREME RIGHT BACKGROUND), POSTAL TELEGRAPH, AND HOME LIFE INSURANCE COMPANY (WITH POINTED BLACK TOP), SITE OF WOOLWORTH BUILDING ACROSS BROADWAY FROM POST OFFICE.

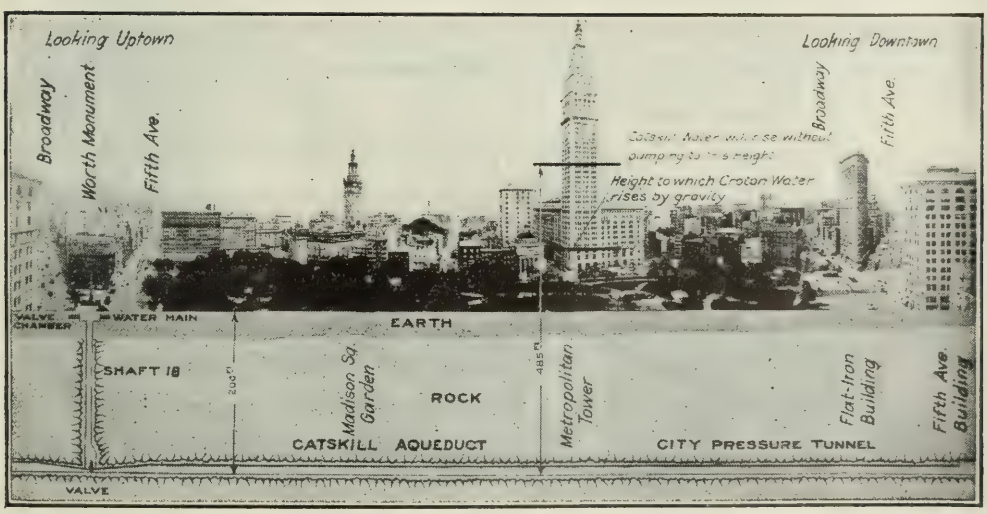

FIG. 73.—TALL BUILDINGS AT MADISON SQUARE; PANORAMA SHOWING CITY PRESSURE TUNNEL.

heavy pieces have given railroad and other engineers hard nuts to crack. Heating, ventilating, pumping, lighting, elevator and other mechanical equipment in these great buildings frequently mount in cost high in the hundreds of thousands of dollars, and exceed in capacity and power the public service plants of small towns. About fifty years ago the first I-beams rolled commercially were used in the Cooper Union and a few other New York buildings. They were of iron, and these buildings were the first in the world to have rolled iron structural members in their frames. Now thousands of tons of structural steel are used in the City's new buildings every year.

With the development of "steel-cage" and fire-proof construction the height and weight of buildings have increased rapidly. To support such buildings satisfactorily on Manhattan Island, new types of foundation became necessary, especially south of 14th Street, where the rock is at considerable depth below tide level and is overlaid with sand and gravel which admit water freely. Pneumatic caissons were developed to meet this need. Both cylindrical and rectangular shapes have been used. By this means exterior walls and columns and interior columns are supported on the bed-rock. At least one building, the Stock Exchange, has its subterranean stories enclosed and made possible by a coffer-dam of connected rectangular caissons. Large pneumatic caissons have been used also in sinking to rock some of the shafts for railroad and water-supply tunnels. Caissons are variously constructed of timber, of steel, and of reinforced concrete. Those used for foundations are commonly filled solid with Portland cement concrete. For a long time wooden piles have been used extensively, but several cases of serious trouble have developed recently from the decaying of the upper portions of such piles in sections of the City where the ground-water has been lowered by the more thorough drainage incident to extensive underground construction, especially sewers and subways. These and other circumstances are leading to the frequent use of concrete piles.

The following are some of the well-known, down-town, business buildings which have caisson foundations sunk through sand and water to bed-rock; in each case the depth of the foundation below the street level is given in feet: No. 42 Broadway, 53; Whitehall Building, 55; Bankers' Trust Building, 69; Trinity Building, 82; Fidelity Building, 83; United States Realty Building, 85; Singer Tower, 92; Liberty

Tower, 94; Western Union Building, 101; Woolworth Building, 125; Municipal Building, 150, this last being the deepest in New York City.

Telephone and Telegraph Systems.

But for its engineering activities, modern New York could not have come into being, and could not be nourished and maintained. Without telephones, engineering, business, and social activities alike, would be impracticable on the present scale. Sixty-two exchanges, 500 000 telephones, 6 000 men, and 7 500 girls, making 2 000 000 connections daily, 1 200 000 miles of underground wire, and countless intricate appurtenances, make up New York's distance-speaking system. It has been developed in 35 years, is growing at the rate of 100 telephones a day, and is constantly being improved. The New York company alone is connecting as many instruments as there are in all Great Britain and Germany combined. New York's long-distance telephone, telegraph, cable, and wireless systems give immediate communication with all points on the globe having electrical means for transmitting information.

Telephone, telegraph, and practically all other, wires are placed in ducts under the street pavement, except in some outlying districts where this improvement has not yet been effected and the ugly poles still disfigure the streets.

Electric Light and Power.

In ever-increasing measure, electricity is illuminating the streets, houses, shops, and factories of the City, and supplying the power for its industry. To meet this demand some notable power-generating stations have been built by the Edison companies, from which radiate miles of conduits through all the boroughs. Another station, which when completed will be among the largest in the world, is in course of construction. Fully inter-connected, no accident in any one station can interfere with the current supplied to any part of the district. The perfecting of the steam turbine has wrought startling changes in some big power-stations, and will force similar changes in others. Waterside Station, having a capacity of 500 000 h.p., burning 2 000 tons of coal daily, and supplying 5 000 000 lamps, is the largest of its kind in the world. The Edison system, covering 21 sq. miles, embraces 33 sub-stations and has 5 000 employees, who make more than 5 000 000 telephone calls a year in conducting their work. It

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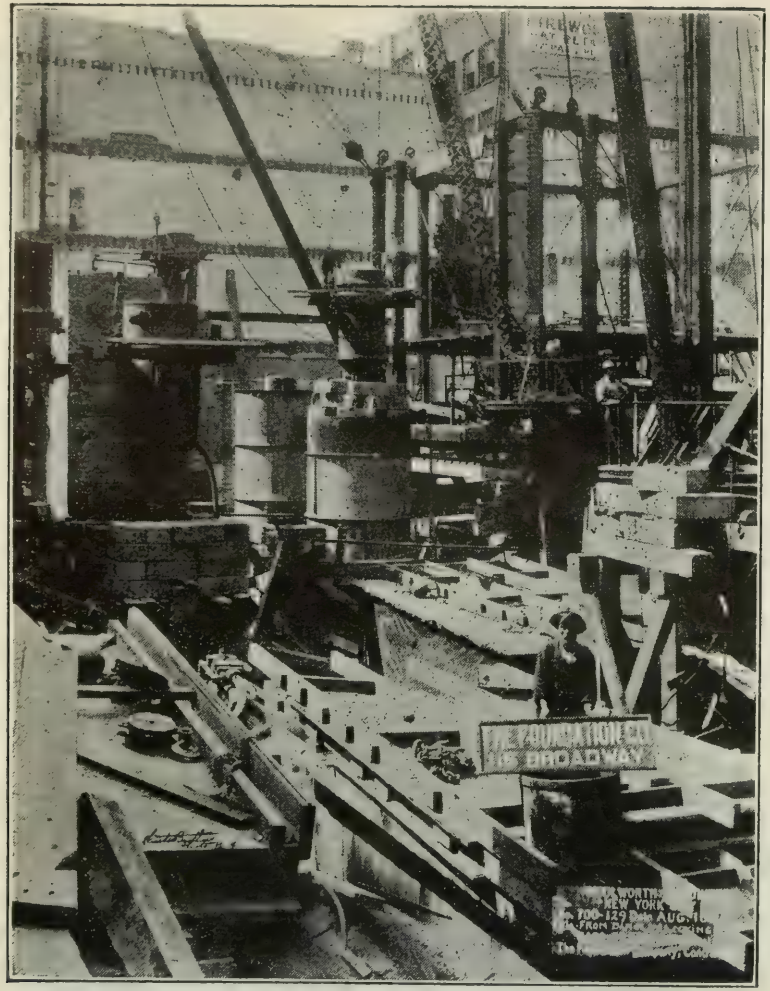

FIG. 74.—WOOLWORTH BUILDING FOUNDATIONS: PORTION SHOWING CYLINDRICAL PNEUMATIC CAISSONS WITH AIR LOCKS; ALSO SQUARE ERECTING TRAVELER WITH DERRICK BOOMS

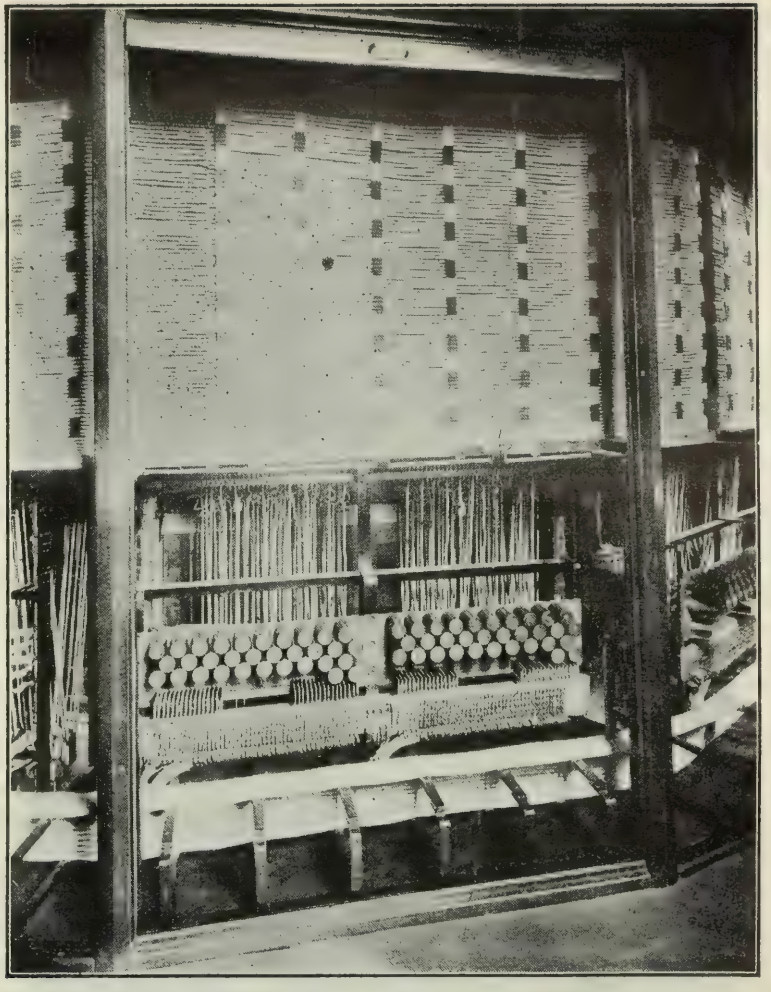

FIG. 75.—NEW YORK TELEPHONE SYSTEM: REAR OF AN OUTGOING SWITCH-BOARD IN A LARGE EXCHANGE, SHOWING ONE SECTION OR MULTIPLE. THERE ARE 10 200

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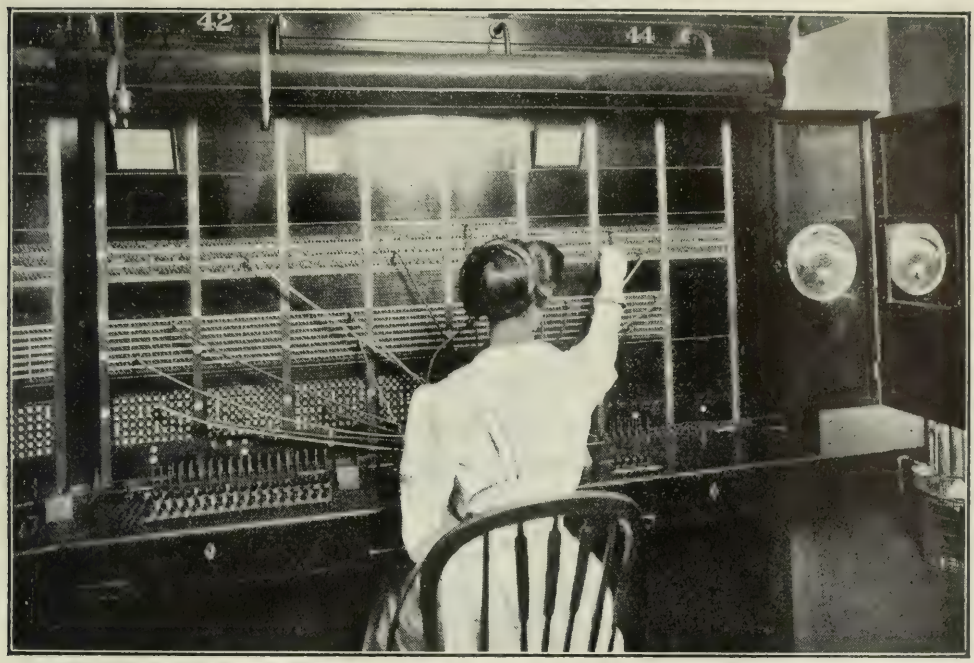

FIG. 76.—NEW YORK TELEPHONE SYSTEM: A RECEIVING SWITCH-BOARD IN A MODERN EXCHANGE. AT THIS BOARD TRUNK-LINE CONNECTIONS ARE MADE. ONLY ONE OF TWENTY-ONE SECTIONS OR MULTIPLES IS SHOWN. OPERATORS AT THESE BOARDS DO NOT TALK WITH SUBSCRIBERS, BUT ONLY WITH OTHER EXCHANGES.

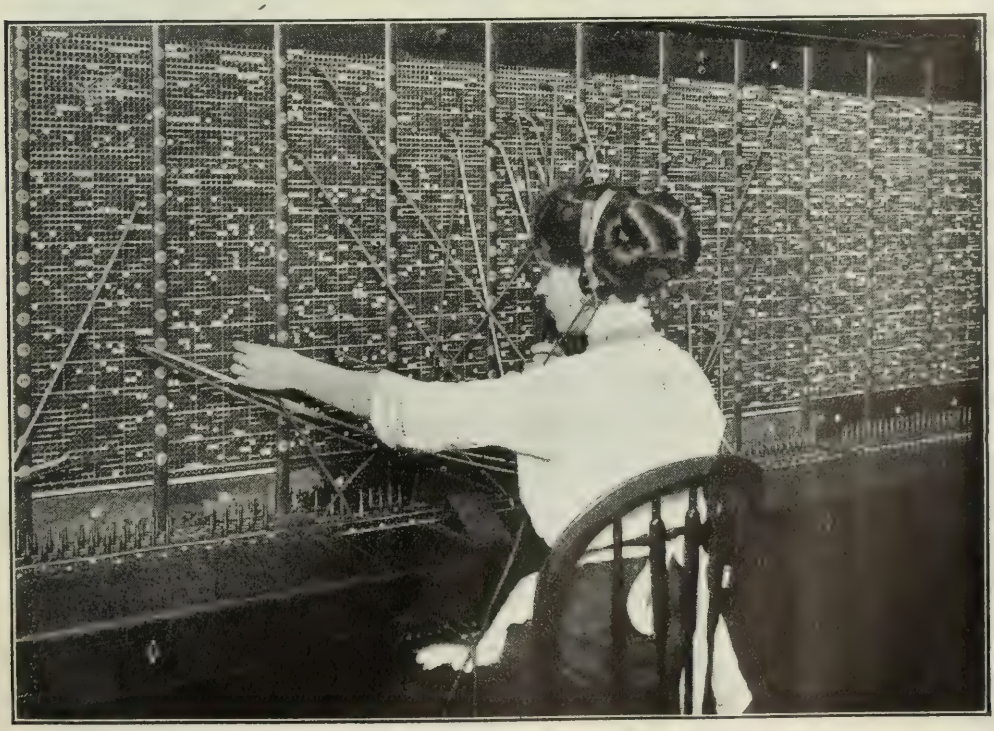

FIG. 77.—NEW YORK TELEPHONE SYSTEM: AN OUTGOING SWITCH-BOARD IN A LARGE EXCHANGE. AT THIS BOARD CONNECTIONS BETWEEN TRUNK LINES AND SUBSCRIBERS' WIRES ARE MADE. OPERATORS AT THESE BOARDS TALK WITH THE PERSON ANSWERING THE 'PHONE.

supplies 92 000 customers. So reliable has been the service that in 30 years the current has left the mains only twice, once for a few hours because a great fire nearly destroyed the original station, and again for a few moments when changing to the first modern station.

Gas.

Gas is used extensively for lighting, cooking, and industrial purposes. Many plants, large and small, for its manufacture, are scattered about, but most interesting is the one in course of development at Astoria, in Queens Borough, which is ultimately to have a capacity of 500 000 000 cu. ft. per day, making it the largest in the world. These works already include the largest gas-holder in the world, with a capacity of 15 000 000 cu. ft., with a reinforced concrete tank 300 ft. in diameter, 57 ft. deep; and rapid coal unloading towers and bridge having a capacity of 2 000 tons in 10 hours; the capacity of the bucket is 7 tons, and, with its trolley, it weighs 20 tons; also in the group are brick coke pockets having a capacity of 33 000 cu. ft., to which coke is delivered directly from the retorts, and thence into railroad cars. The largest gas mains beneath the streets are 60 in. in diameter. An innovation is the making of the cast-iron pipe joints with lead wool, or shredded lead, caulked with pneumatic hammers. Many streets and parks are illuminated by gas, Welsbach or similar mantles being used extensively. All told, there are 3 640 miles of gas mains. There are five gas companies supplying New York City; the largest of these, the Consolidated Gas Company, was formed in 1884 by combining six companies.

For conveying illuminating gas from the works at Ravenswood, on the western extremity of Long Island, to Manhattan, a tunnel beneath the East River and Blackwells Island was projected in the early Nineties. Begun in 1892 and completed in 1894, this tunnel, 2 106 ft. long, was the first to be driven beneath the East River. It is a little more than 10 ft. in diameter through soft ground, and 8½ ft. high and 10½ ft. wide in sound rock, large enough for two 3-ft. and one 4-ft. gas mains. It was driven from a shaft 139 ft. deep at 71st Street, Manhattan, and another of the same depth on the Ravenswood shore. The minimum thickness of roof is 40 ft. beneath the west channel and 100 ft. beneath the east channel. Superficial examinations and some pipe soundings led to the expectation of solid rock for the whole length,

but several wide zones of soft, decomposed, and water-bearing rock were encountered under the channels, requiring compressed air up to 48 lb. per sq. in. and tunneling shields, with segmental cast-iron lining. Gas was first delivered through the tunnel on October 15th, 1894. A second tunnel is now in progress, in the same vicinity, for additional gas mains, connecting Astoria, on the Long Island shore, with East 111th Street, Manhattan.

Parks.

Father Knickerbocker's children delight to play as well as to work. New York has many public parks, large and small, aggregating 7 580 acres; many public playgrounds, and innumerable private amusement resorts. Most notable among the parks are Central and Riverside Parks, Manhattan; Prospect Park, Brooklyn; Van Cortlandt, Pelham Bay, and Bronx Parks, in The Bronx; and Forest Park, in the Borough of Queens.

Central Park is, beyond question, more widely known than any other park in America. Its site, when purchased in 1856, was waste land occupied by "squatters' shanties," slaughter houses, bone-boiling works, and similar ugly and unsavory buildings. Its present valuation is more than sufficient to offset one-quarter of the City's total public indebtedness.

Bronx Park contains famous zoological and botanical gardens, with many collections of rare specimens. Although possessing some natural interest, all these parks owe their present attractiveness largely to skillful landscape engineering. In Battery Park, at the south tip of Manhattan, there is an unusually instructive and well-arranged aquarium.

An extensive system of boulevards leads through panoramas of park and city, mountain and marsh, river and ocean. Begun many years ago, it is still in course of development, and, owing to the automobile, is being projected even beyond the City's confines. In connection with its water supplies, the City owns many thousands of acres of land, which can ultimately be made enjoyable as an outer park system. The Bronx Parkway Commission, organized in 1907, has taken the initial steps toward rescuing the Bronx River and its banks, from Bronx Park to the Kensico Dam. Along this stream a scenic drive and continuous park are gradually to be developed to connect the parks and boulevards within the City with the outlying reservoir

FIG. 78.—NEW YORK EDISON COMPANY'S WATERSIDE POWER STATION, MANHATTAN. VIEW FROM EAST RIVER.

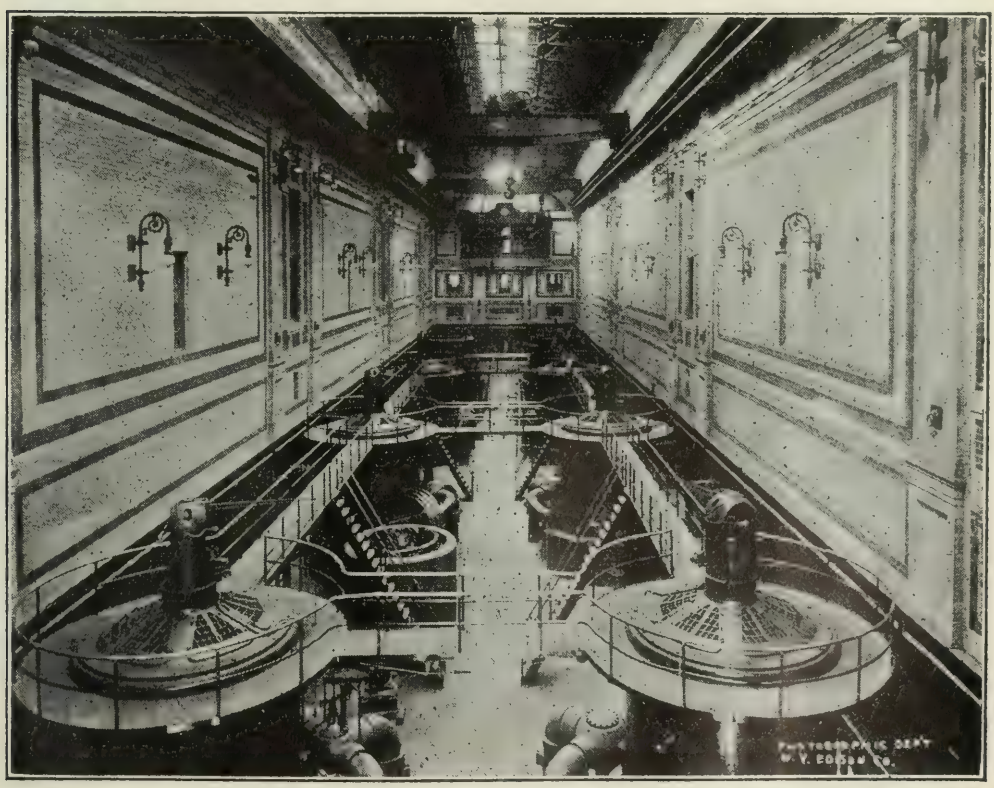

FIG. 79.—NEW YORK EDISON COMPANY'S WATERSIDE POWER STATION. INTERIOR OF TURBO-GENERATOR ROOM.

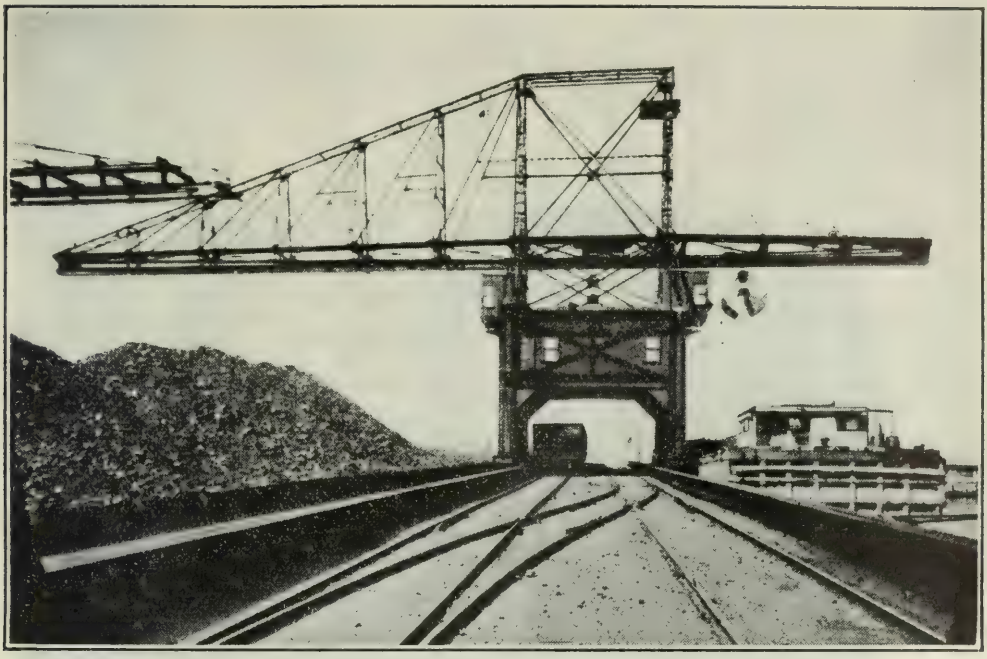

FIG. 80.—CONSOLIDATED GAS COMPANY: TRAVELING TOWER FOR UNLOADING COAL AT ASTORIA.

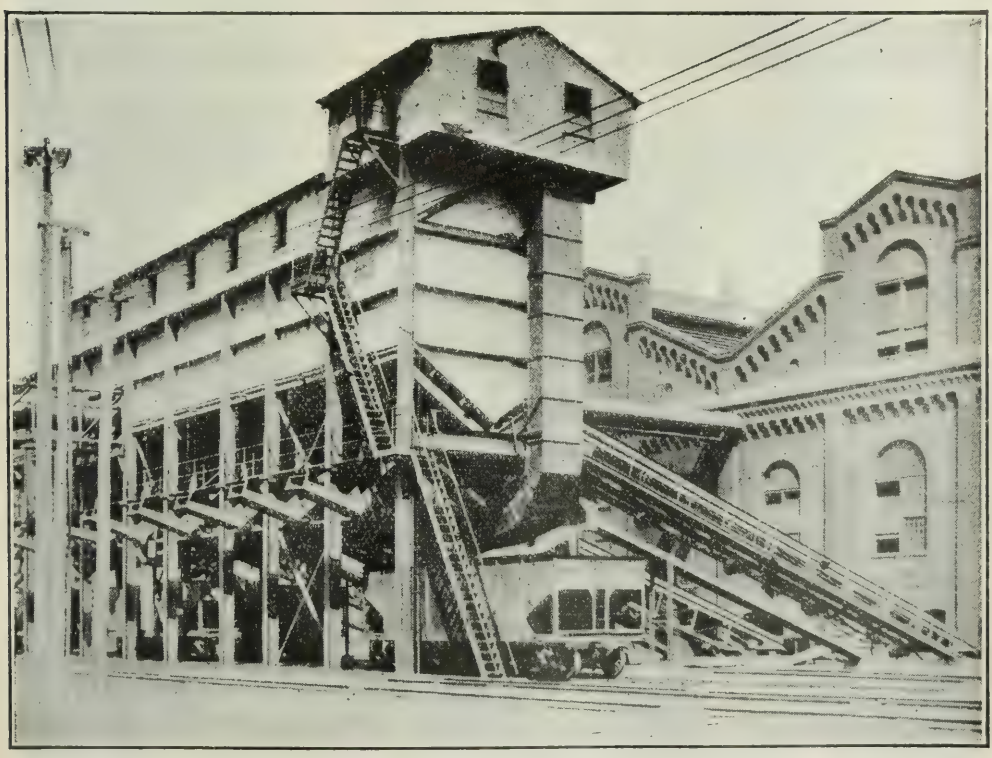

FIG. 81.—CONSOLIDATED GAS COMPANY: COKE PLANT AT ASTORIA.

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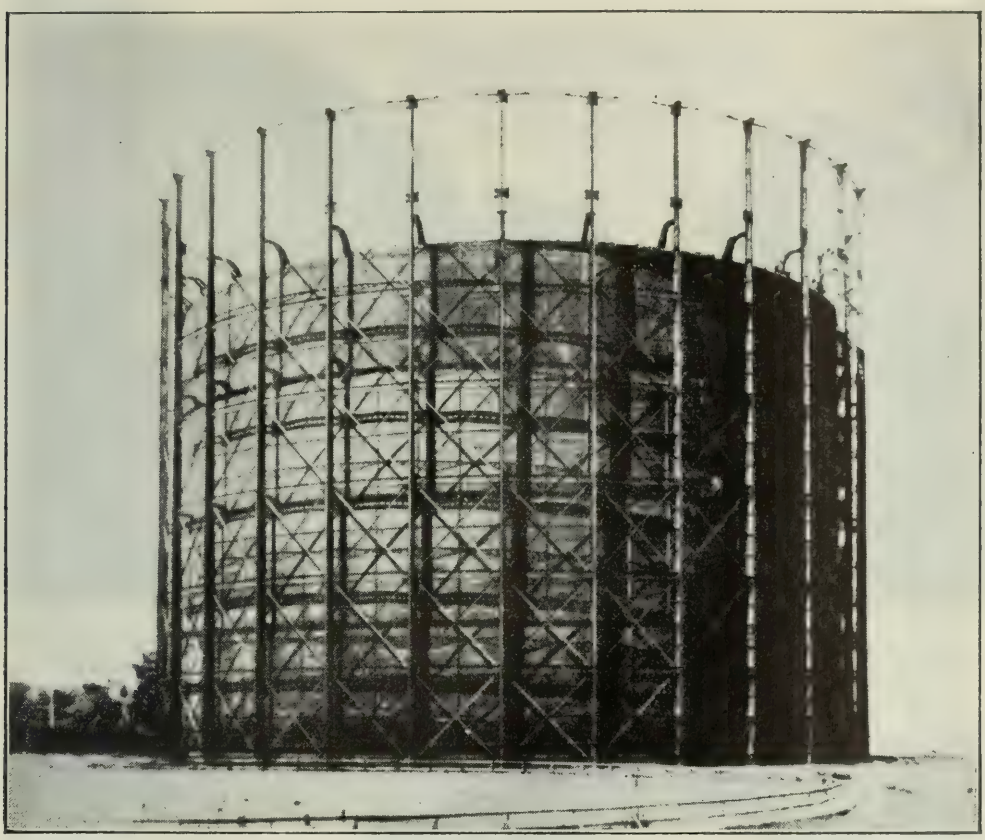

FIG. 82.—CONSOLIDATED GAS COMPANY: LARGEST GAS HOLDER IN THE WORLD.

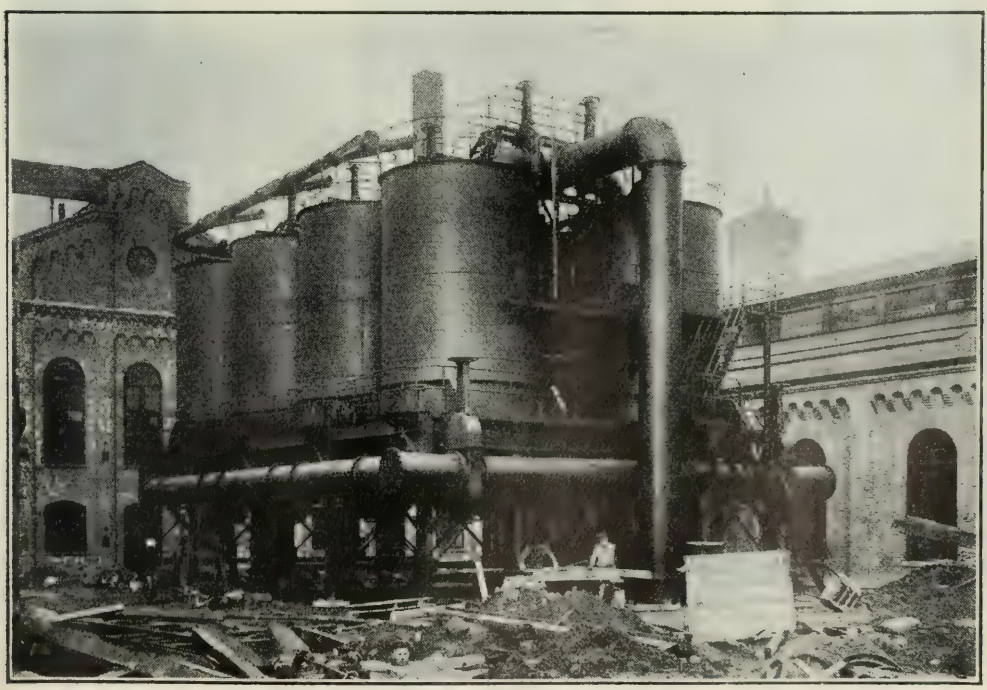

FIG. 83.—CONSOLIDATED GAS COMPANY'S SCRUBBERS AT ASTORIA, FOR PURIFYING GAS.

FIG. 84.—CENTRAL PARK; LOOKING TOWARD THE EAST DRIVE, NEAR 79TH STREET TRANSVERSE ROAD. ALMOST TOTAL CONCEALMENT OF THE BUILDINGS ALONG ADJOINING STREETS HAS BEEN ACHIEVED.

FIG. 85.—CENTRAL PARK, SOUTHEAST CORNER, 59TH STREET AND FIFTH AVENUE, FROM THE PROMONTORY, SHOWING THE LAKE. THIS IS THE MAIN ENTRANCE TO THE EAST DRIVE OF THE PARK. THE PLAZA HOTEL IS IN THE MIDDLE BACKGROUND, THE SAVOY AND HOTEL NETHERLANDS, IN ORDER, TO THE LEFT.

lands and the beautiful residential districts of Westchester County and the regions beyond. Ultimately, this system will be linked with the State boulevards and park reservations leading to the Catskill Mountain water-sheds. Realization of these schemes, now well in hand, will perfect for the New York metropolitan district and all the lower Hudson Valley towns a park system of such beauty, extensiveness, and accessibility as but few great communities can possess.

Sewerage and Sewage Disposal.

In the important field of sanitary engineering, New York must confess to being laggard, at least in so far as the disposal of its sewage is concerned. With so extended a water-front, bordering such vast bodies of water, it has been easy to prolong the sewers to the shores and let the crude sewage flow at innumerable points into the rivers and harbor. Even communities situated miles inland, either by trunk sewers directly, or indirectly through the rivers, have used this same simple method of getting rid of their water-borne filth. With increase of population this practice has caused nuisances and menaced health, especially through the oyster and other shell-fish beds in the outlying bays, and the public bathing establishments along the water-front, where 2 000 000 baths are taken annually. There are 3 000 miles of sewers in the forty cities of the metropolitan district, 1 700 miles being in New York City. A few small sewage treatment plants have been indifferently operated for a number of years. Others are being constructed and better methods adopted.

The subject is now receiving, from the Metropolitan Sewerage Commission of New York City, that broad study which is needed; this study by experts will aid materially in planning an adequate and comprehensive system for the whole basin draining into New York Bay. Five years have already been spent in extensive investigations of the condition of the harbor, of tidal movements, of currents, of population, topography, existing sewer systems, and of many other phases of the problem of collecting sewage and storm-water and properly disposing thereof, for the whole interstate metropolitan drainage basin. In these studies the Commission has from time to time had the assistance of distinguished English and American sanitarians. Tentative plans for definite engineering works have been drawn and approximate estimates of cost made. The Commission has found that the methods by which sewage is disposed of demand immediate and far-reaching

improvement. It is impracticable to construct a system or systems of conduits to collect the sewage from the whole district and convey it to one place for disposal. The district must be subdivided, and various methods of treatment and discharge, suitable to local requirements, must be adopted.

Revision of existing local systems and the construction of large trunk sewers and treatment works will probably be undertaken in a future not many years remote.

Some of the administrative and physical obstacles in the path of improved sewerage are suggested by the following facts: There are eighty communities concerned, and they are situated in two independent, sovereign States. A suitable metropolitan sewerage district would comprise an area of about 700 sq. miles, practically all within 20 miles of New York's City Hall, of which about one-half is in each of the States of New York and New Jersey. The Federal Government also would be a party to any comprehensive scheme. There are many diverse and conflicting, commercial, manufacturing, political, and recreative interests involved. Of the proposed district, 180 sq. miles are bodies of water ranging in depth from about 5 to 150 ft., with maximum velocities of flow ranging from 2 to 10 ft. per sec., the latter occurring in parts of the East River.

The densities of population per square mile were, in 1910: In New Jersey, on the Rahway River water-shed, 253; in the Joint Outlet Sewer district, 1 700; in the Passaic Valley, 5 800; in the New York Boroughs, Queens, 2 900; Richmond, 1 700; The Bronx, 13 400; Brooklyn, 25 500; and Manhattan, 112 300. Some of the most congested areas border on parts of the harbor naturally least able to purge themselves of the inflowing sewage. In 1910, the total sewage was about 635 000 000 gal. daily, and the Metropolitan Sewerage Commission has estimated that by 1940 the volume will be doubled. Through The Narrows there flow at each tide approximately 12 000 000 000 cu. ft., the excess of water moving seaward over that returning being roughly 1 300 000 000 cu. ft.

Although backward in the treatment of its sewage, Greater New York has built some remarkable sewers, having many miles 10 ft. or greater in diameter, in Brooklyn and The Bronx principally. Some of the largest sewers are 15 ft. in diameter. The "combined" system of sewerage is almost universal.

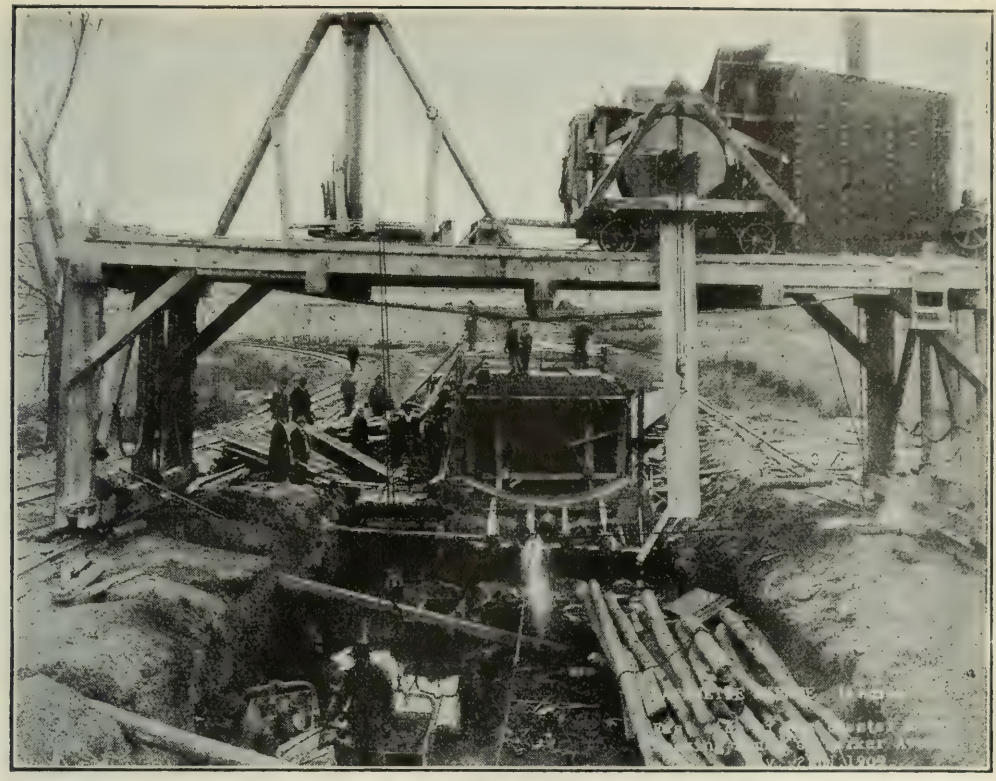

FIG. 86.—METHOD OF CONSTRUCTING A LARGE OUTFALL SEWER IN THE BRONX. REINFORCED CONCRETE SUPPORTED BY WOODEN PILES.

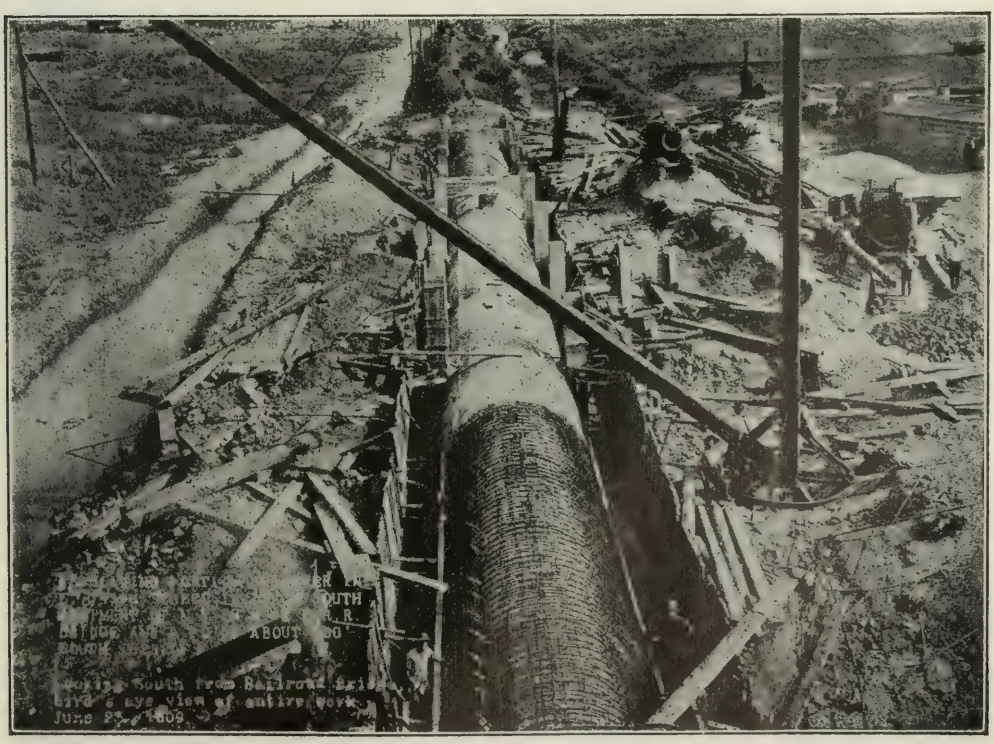

FIG. 87.—ARCHED REINFORCED CONCRETE SEWER IN THE BRONX. VIEW SHOWS METHODS OF CONSTRUCTION, ALSO VERTICAL CONCRETE-ENCASED TILE PIPES FOR FUTURE HOUSE CONNECTIONS.

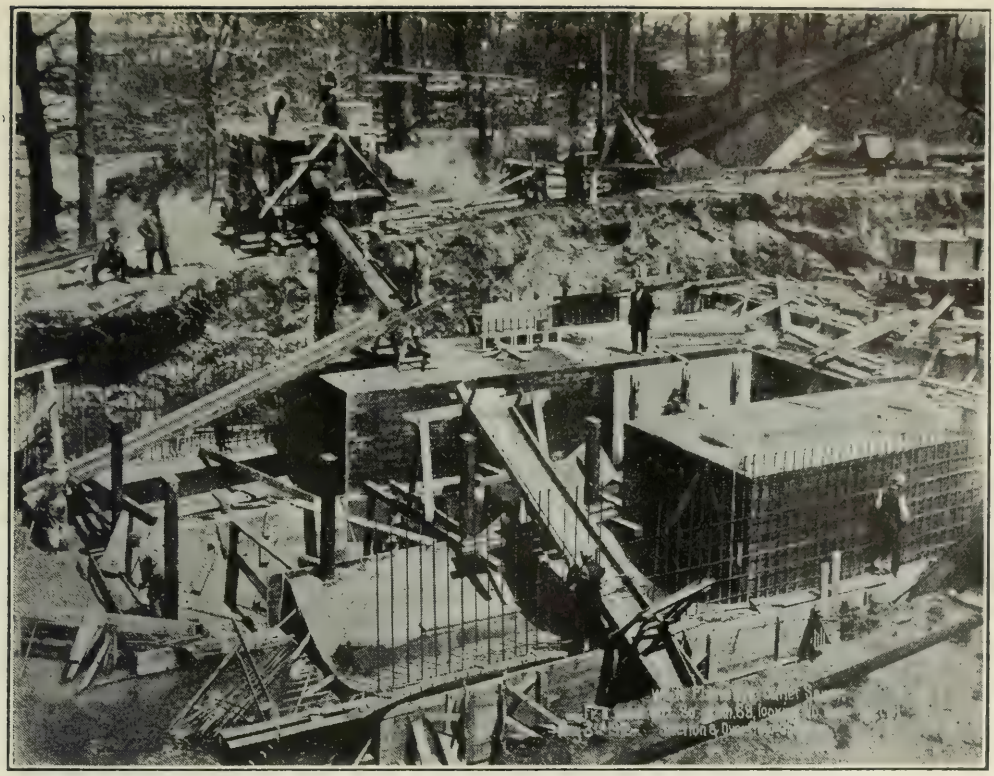

FIG. 88.—OVERFLOW CHAMBER IN LARGE OUTFALL SEWER IN THE BRONX. CONCRETE FORMS, STEEL REINFORCEMENT, PARTLY CONSTRUCTED INVERTS, AND CONTRACTOR'S EQUIPMENT ARE SHOWN.

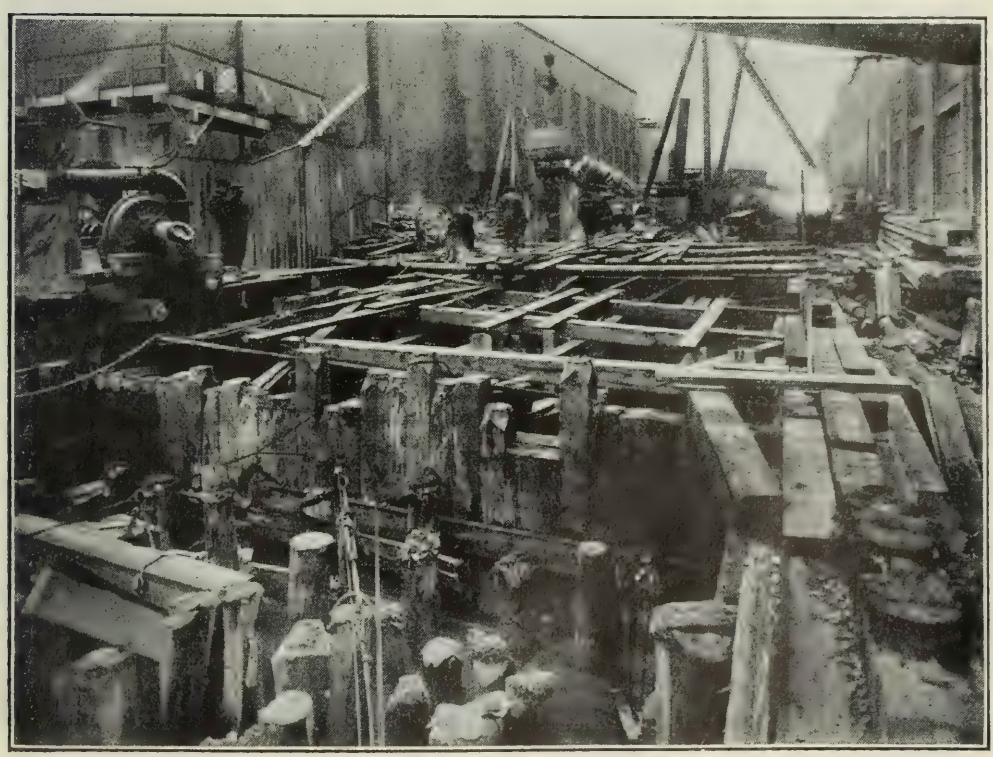

FIG. 89.—GOLD STREET SEWER OUTLET, AT EAST RIVER, BROOKLYN, UNDER CONSTRUCTION.

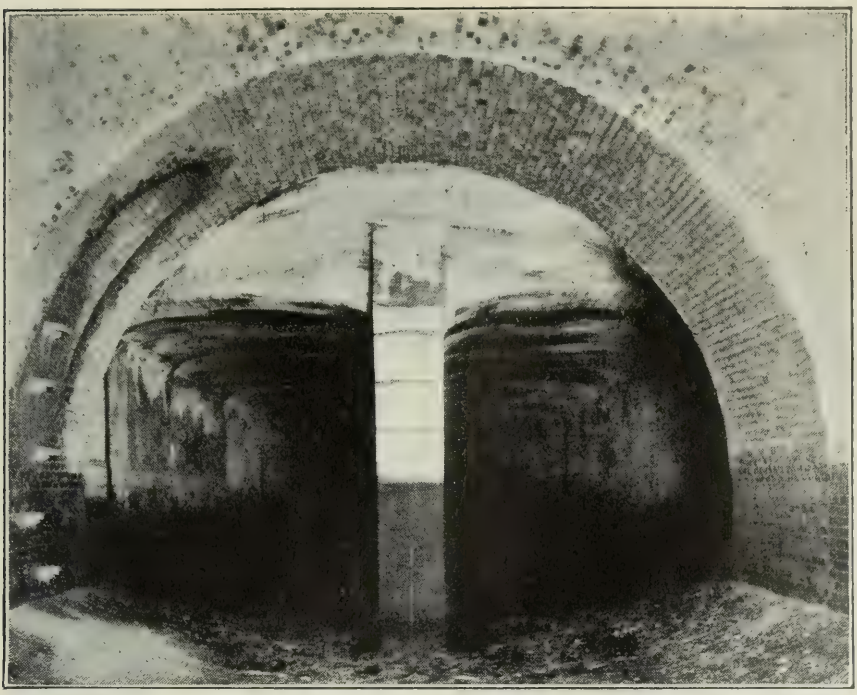

FIG. 90.—SEWER IN ST. NICHOLAS AVENUE, BROOKLYN. SINGLE SECTION 15 FEET IN DIAMETER. VIEW SHOWS JUNCTION OF SINGLE-BARREL BRICK AND CONCRETE TUBE, WITH DOUBLE-BARREL REINFORCED CONCRETE TUBES.

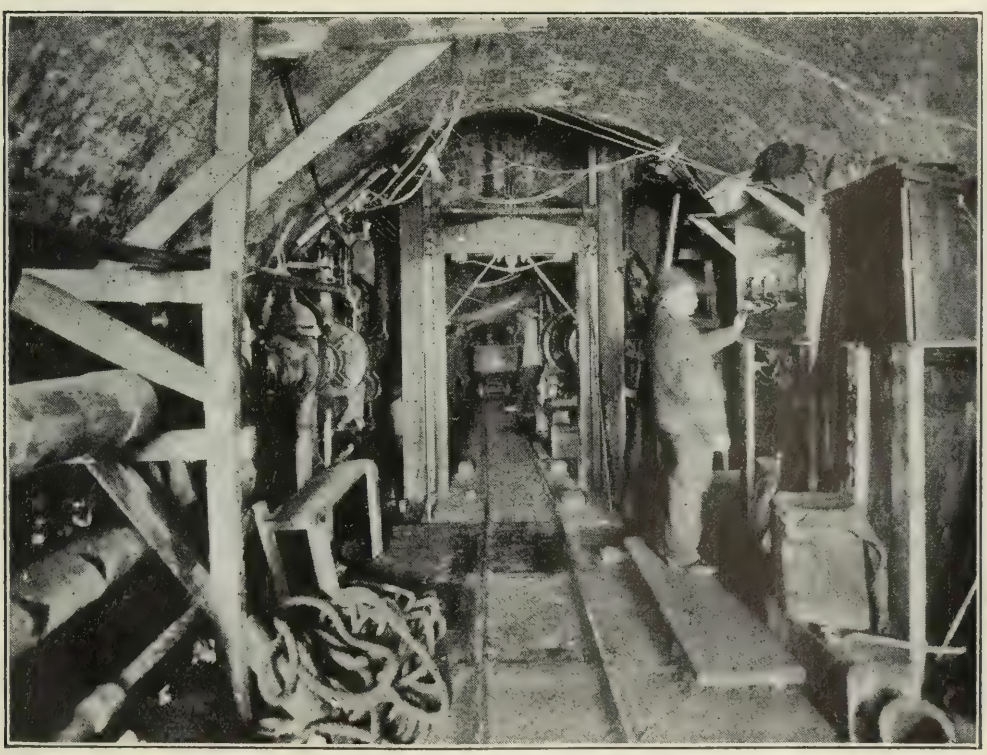

FIG. 91.—VIEW IN FLUSHING TUNNEL, BROOKLYN, AT FOOT OF SHAFT, DURING CONSTRUCTION.

1983

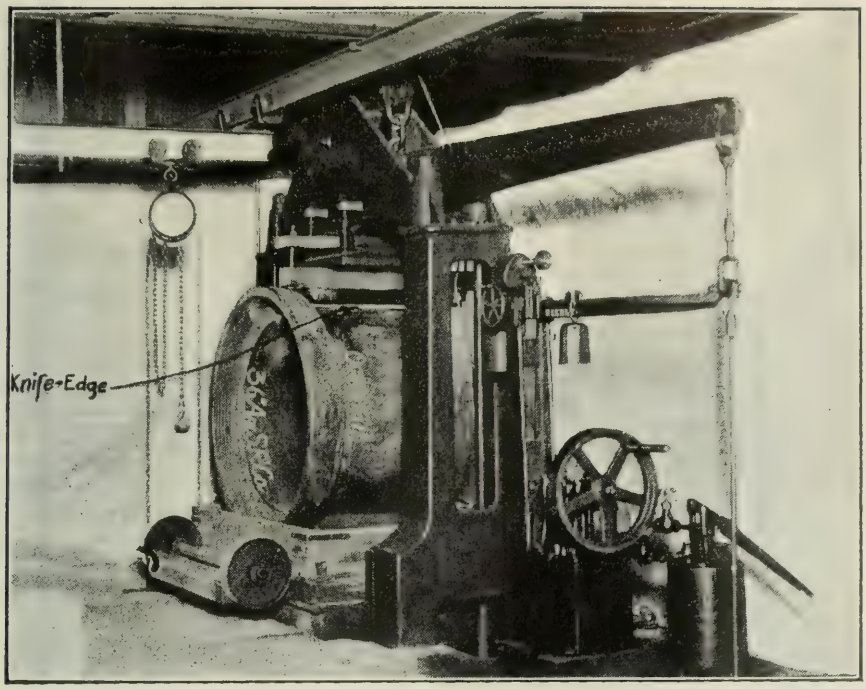

FIG. 92.—TESTING MACHINE, IN PIPE YARD, BROOKLYN. TESTING CRUSHING STRENGTH OF VITRIFIED CLAY SEWER PIPE.

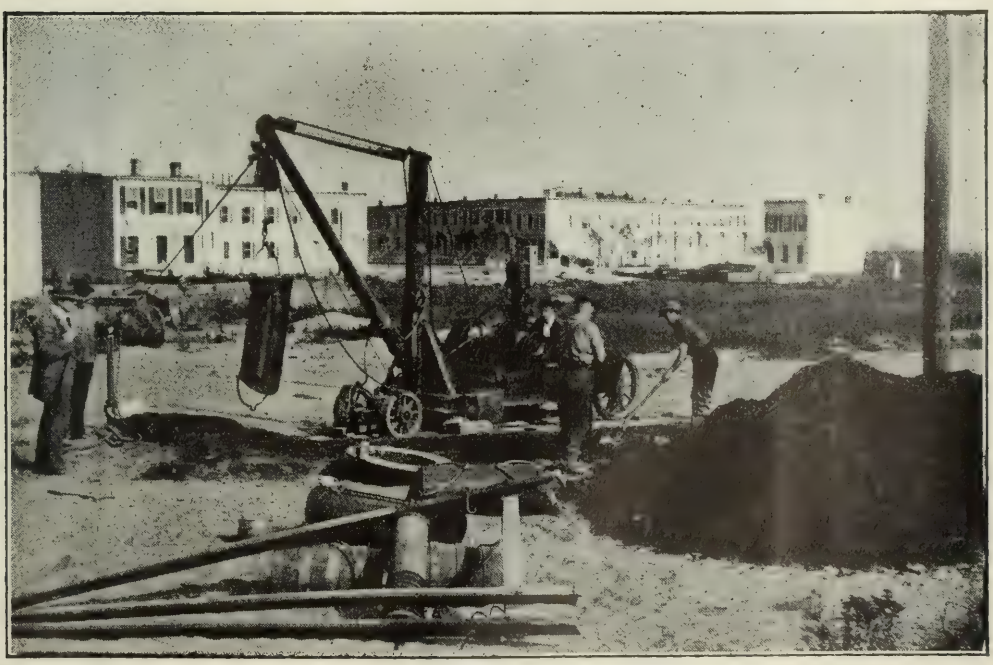

FIG. 93.—TRAVELING DERRICK, WITH GASOLINE ENGINE. DESIGNED AND USED BY THE BROOKLYN SEWER BUREAU FOR CLEANING LARGE SEWERS.

1794

PLATE LXIV.
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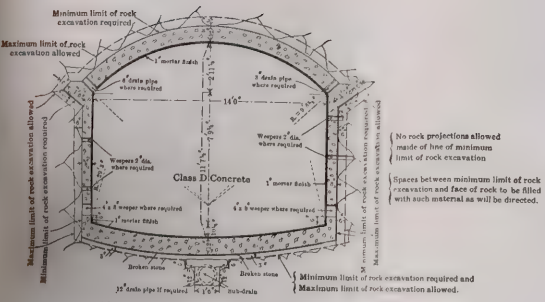
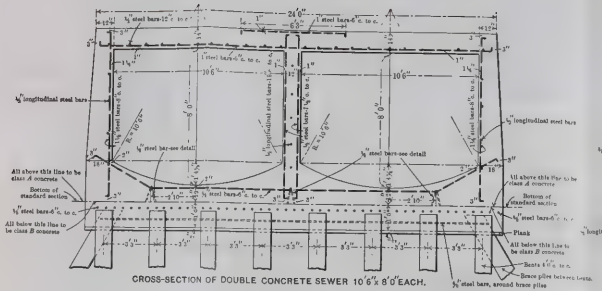
1784A.

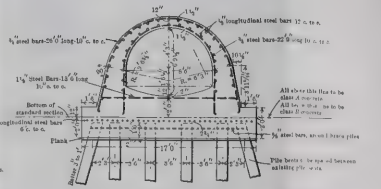
BOROUGH OF THE BRONX
CITY OF NEW YORK
EXAMPLES OF LARGE SEWERS.


Minimum limit of rock excavation required
Maximum limit of rock excavation allowed
Class D Concrete
Weepers 2" dia. where required
Broken stone
Sub-drain
No rock projections allowed inside of line of minimum limit of rock excavation
Spaces between minimum limit of rock excavation and face of rock to be filled with such material as will be directed.
Minimum limit of rock excavation required and Maximum limit of rock excavation allowed.

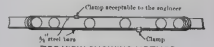
REQUIRED CROSS SECTION IN TUNNEL


All above this line to be class A concrete
All below this line to be class B concrete
Bottom of standard section
Plank
Brace piles between bents

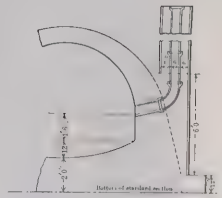
CROSS-SECTION OF DOUBLE CONCRETE SEWER 10'6" x 8'0" EACH.


All above this line to be class A concrete
All below this line to be class B concrete
Bottom of standard section
Plank
Brace piles between bents

CROSS-SECTION OF CONCRETE SEWER 8'6" x 6'9½"

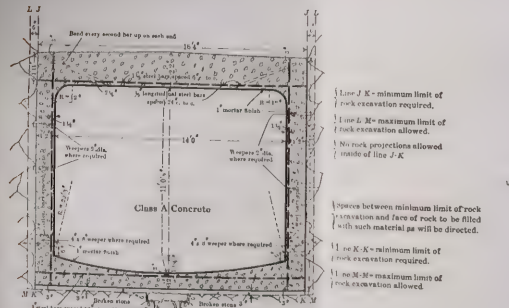


TOP VIEW SHOWING METHOD OF SECURING BRACE PILES.

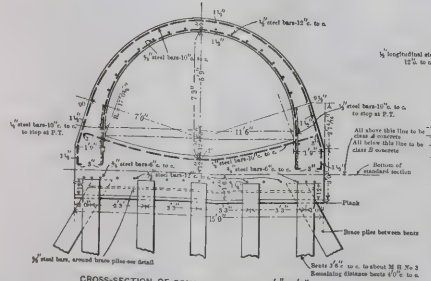


ELEVATION
ARCH SECTION
SPUR PIPE AND RISER
IN CONCRETE FOR HOUSE CONNECTIONS.

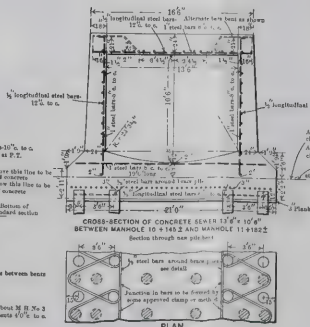
Risers for house connections to be carried to a point 2 ft. below the fin stool grade of the embankment


Class A Concrete
Weepers 2" dia. where required
Broken stone
Sub-drain
Line J-K - minimum limit of rock excavation required.
Line L-M - maximum limit of rock excavation allowed.
No rock projections allowed inside of line J-K
Spaces between minimum limit of rock excavation and face of rock to be filled with such material as will be directed.
Line K-K - minimum limit of rock excavation required.
Line M-M - maximum limit of rock excavation allowed.

TYPICAL CROSS-SECTION IN OPEN ROCK CUT.

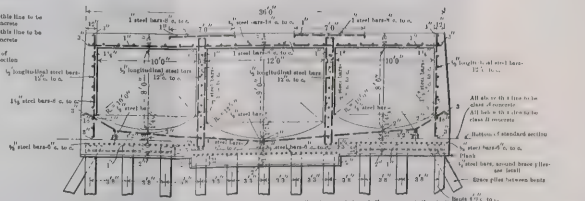

All above this line to be class A concrete
All below this line to be class B concrete
Bottom of standard section
Plank
Brace piles between bents

CROSS-SECTION OF CONCRETE SEWER 11'6" x 7'3"



CROSS-SECTION OF CONCRETE SEWER 13'6" x 10'6" BETWEEN MANHOLE 10 +145.2 AND MANHOLE 11+182.2

PLAN.

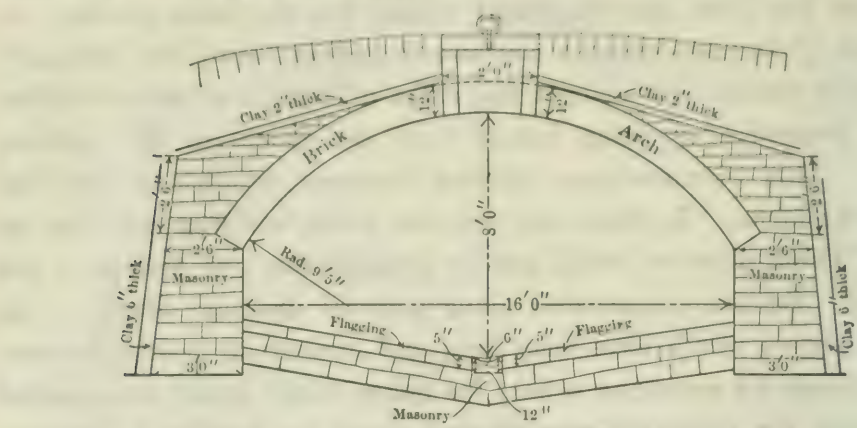

All above this line to be class A concrete
All below this line to be class B concrete
Bottom of standard section
Plank
Brace piles between bents

CROSS-SECTION OF TRIPLE CONCRETE SEWER 10'0" x 8'0", 12'0" x 9'0" AND 10'6" x 8'0".

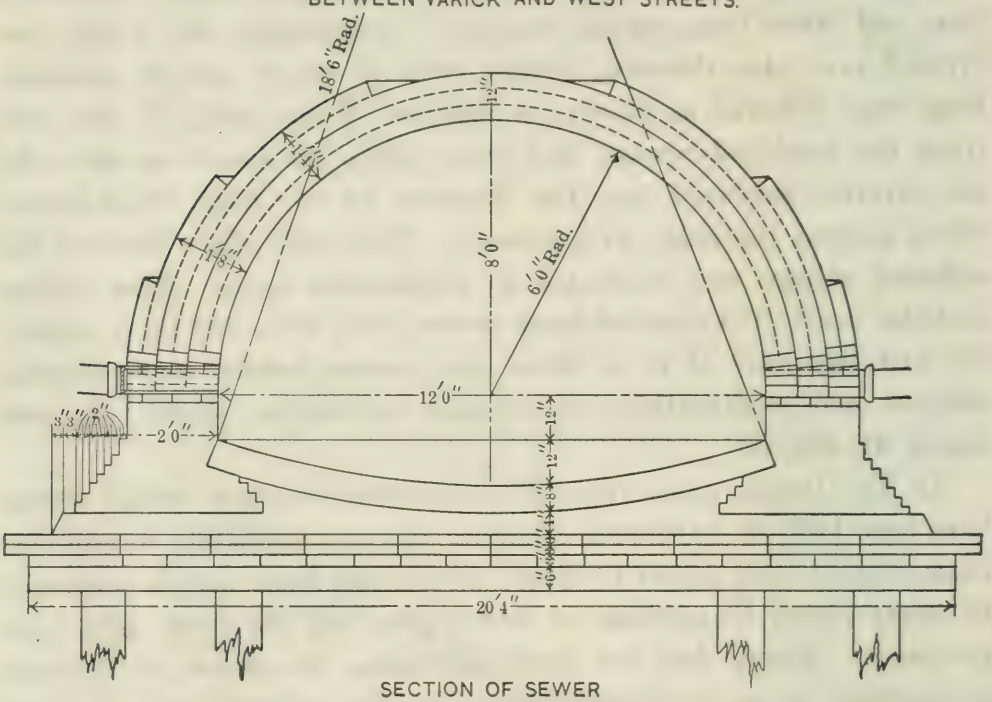
Brooklyn has more than 925 miles of sewers, ranging in diameter from 6 to 187 in., many miles being of very large sizes. The flat topography has been one important reason for the large sewers. In the course of its development southward and eastward the borough's closely built section spread over the ridge of high land which divides its area, instead of growing inland from the shores in the outlying portions. This circumstance injected unusual conditions into the sewerage problems. In these new sections house-building has gone on rapidly. These districts could not be drained into the sewers of the older city. Sewers constructed along natural drainage lines to the southerly shores would have been long and excessively costly, traversing wide areas not requiring sewers for many years; would have needed a long time for construction, delaying relief to developed sections; and would have discharged the sewage toward Coney Island, Gravesend Bay, and other large seaside resorts. Consequently, the region was divided into nine districts, within each of which natural drainage lines were followed as closely as feasible. From some of these districts the combined sewage, and from others the house sewage only, was diverted westward into The Narrows, by two large trunk sewers which pierced the ridge with tunnels. From still other districts the collected sewage was conducted to purification works. This system includes nearly 100 miles of brick sewers from 30 to 180 in. in diameter, and 600 miles of 12 to 24-in. pipe sewers, besides four pumping stations and modifications of existing purification works. It cost nearly $15 000 000.

In The Bronx, many recently constructed principal outfall sewers have been built of reinforced concrete. An approximately rectangular cross-section, with invert of large radius, has been used extensively. In many places foundations of timber piles and platforms have been necessary. Across flat, low lands adjoining the shores, these large sewers have to be built projecting above the natural surface of the ground in order to secure proper hydraulic conditions for the discharge, and avoid having tidal water form a comparatively still basin, reaching far back into the sewer, in which deposition of solids would take place. This latter fault is common in the older sewers of more than one borough. One very large storm-water relief sewer in the western part of the borough (Webster Avenue) was driven for a considerable portion of its length as a tunnel through the rock ridge sepa-

rating the drainage area from the Harlem River, into which it was desired to discharge the water.



SECTION OF SEWER
CANAL STREET
BETWEEN VARICK AND WEST STREETS.



SECTION OF SEWER
110th STREET
FIFTH AVENUE TO HARLEM RIVER.

FIG. 94.—SOME LARGE BRICK AND STONE MASONRY SEWERS IN MANHATTAN BOROUGH.

In the Borough of Queens, where there are still thousands of acres of market gardens outside the several closely-built community centers, large districts are still without sewers, and in others existing sewerage is distressingly inadequate. For this borough, the problems of sewerage and sewage disposal are of unusual complexity. Many large trunk

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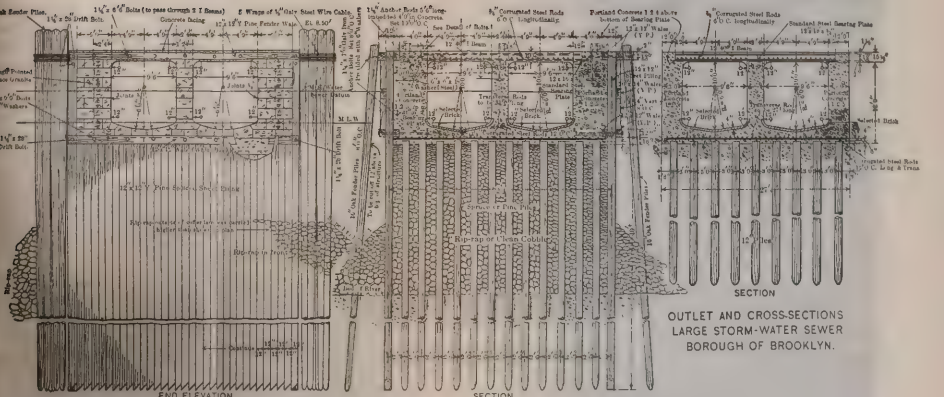
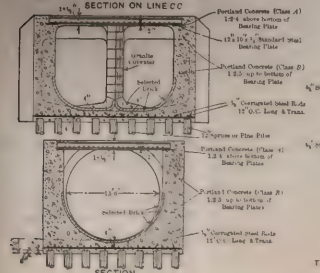
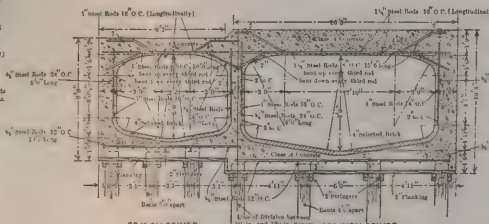
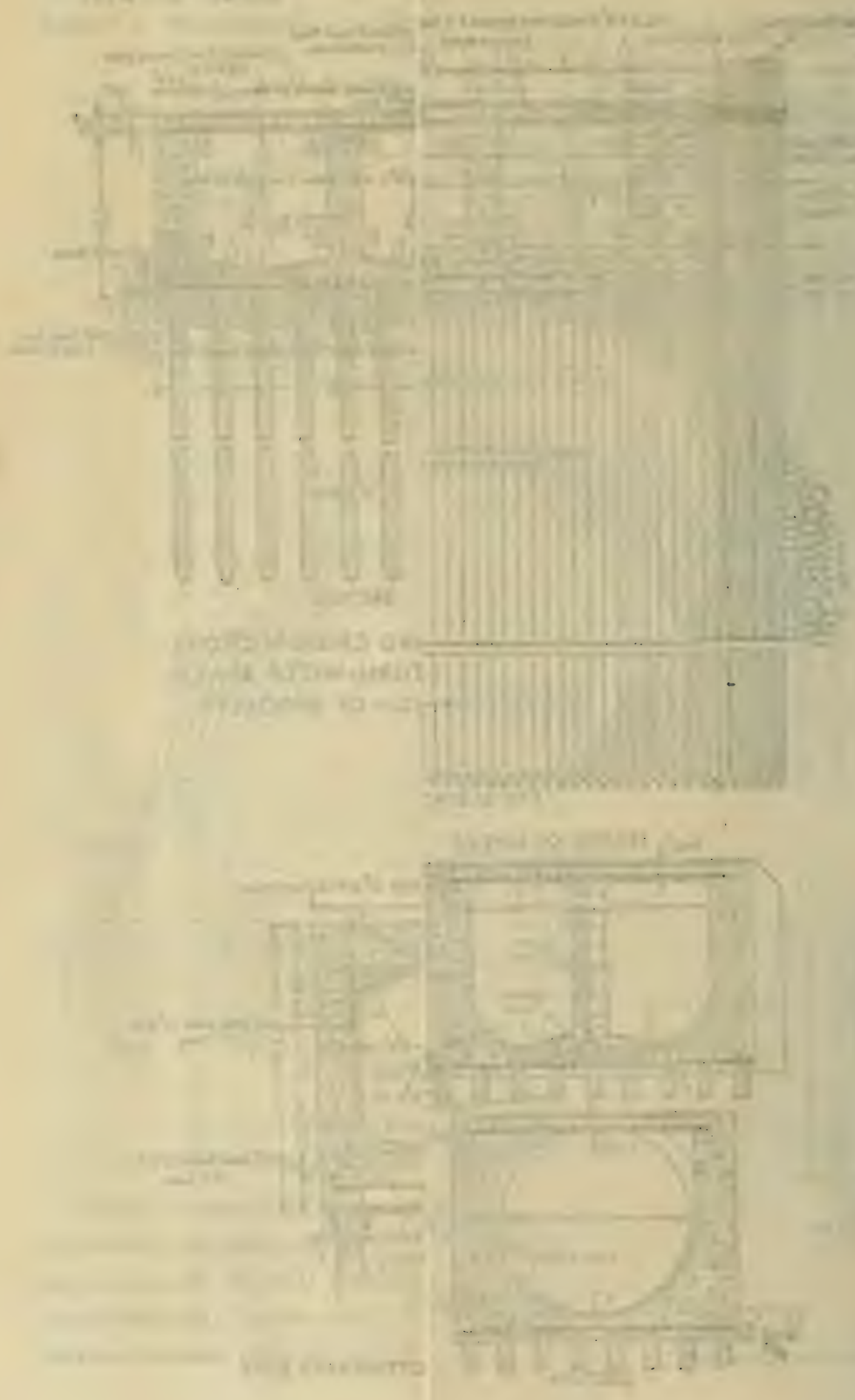


OUTLET AND CROSS-SECTIONS
LARGE STORM-WATER SEWER
BOROUGH OF BROOKLYN.


SECTION ON LINE C-C
Portland Concrete (Class A) 1:2:4 above bottom of Bearing Plate
Portland Concrete (Class B) 1:2:3 up to bottom of Bearing Plate
Selected Brick
Corrugated Steel Rods 12' O.C. Long & Trans.
Portland Concrete (Class A) 1:2:4 above bottom of Bearing Plate
Portland Concrete (Class B) 1:2:3 up to bottom of Bearing Plate
Selected Brick
Corrugated Steel Rods 12' O.C. Long & Trans.
SECTION



THESE SECTIONS USED FOR STORM WATER WHERE COVERING WAS LIGHT AND BOTTOM VERY SOFT



sewers have been built, up to such sizes as 7 by 16 ft., 8 by 12 ft., and 15 ft. in diameter. Some of these connect with large sewers in Brooklyn, emptying into the East River. A number of pumping stations and treatment plants are in operation. With 40% of the area of the greater city, a present average population density of only 4.6 persons per acre in comparison with 39.9 in the neighboring borough of Brooklyn, and greatly improved rapid transit facilities in immediate prospect, it is evident that wisely considered sanitary engineering activities must have a large place in the programme of this borough for several years to come, and that the present 230 miles of sewers must be largely increased.

In Richmond, the sewerage problem, thus far, has been of relatively simple solution, the topography and the location of the borough being favorable.

Water Supply.

Although situated on the shore of a great river, New York is obliged to go many miles for a sufficient supply of potable water. The Hudson is a tidal estuary for 150 miles, and, under usual conditions, its waters are brackish for 70 miles above the City, and, under some conditions, for 85 miles. The Boroughs of Manhattan and The Bronx are supplied from the Croton water-shed, of 360 sq. miles area, and from the upper Bronx River and the Byram River water-sheds, of 22 sq. miles combined area. All these sources have been developed since 1840 by improving seven natural ponds and constructing twelve impounding reservoirs, with an aggregate capacity of 105 000 000 000 gal. For these reservoirs, fifteen large masonry and earth dams have been required. Most famous among them is the New Croton Dam, 2 168 ft. long, with a maximum height of 300 ft. above its lowest foundations. Its waste-weir is 1 000 ft. long, and the whole structure contains 850 000 cu. yd. of rubble, cyclopean, and cut-stone masonry.

Brooklyn gets most of its water from the Ridgewood system of reservoirs, wells, and infiltration galleries, lying east of the City, on Long Island. Queens and Richmond Boroughs get water principally from wells and other small sources, developed for the most part by private companies, some of which have been purchased by the City. All the Brooklyn water is pumped by municipal pumping stations, much of it more than once in order to deliver it at the required eleva-

FIG. 95.—MAP OF NEW YORK CITY'S WATER SUPPLIES.

1789

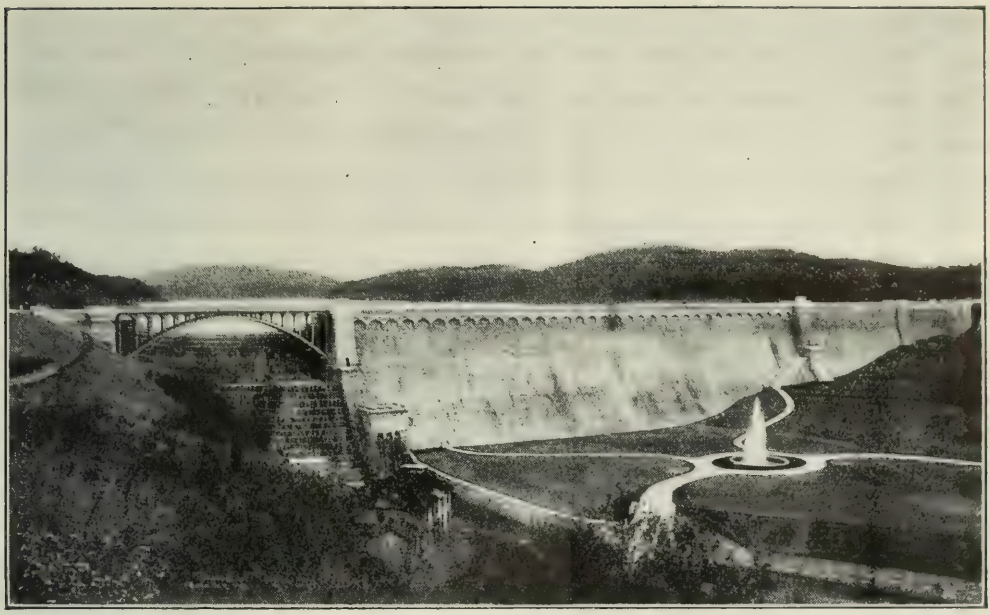


FIG. 96.—NEW CROTON DAM, THE MOST IMPORTANT ON THE CROTON WATER SUPPLY SYSTEM. COMPLETED IN 1905.

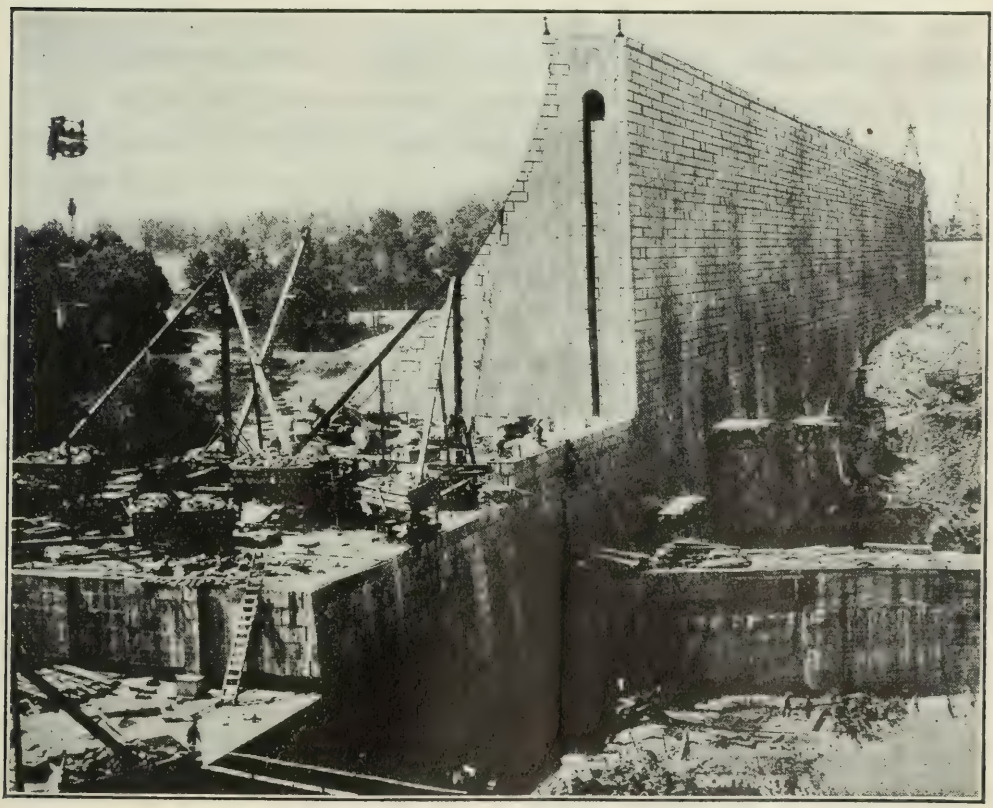


FIG. 97.—OLIVE BRIDGE DAM. MASONRY PORTION 1 000 FEET LONG, 240 FEET HIGH ABOVE DEEPEST FOUNDATION. AT ESOPUS CREEK, ASHOKAN RESERVOIR OF CATSKILL WATER SUPPLY. VIEW SHOWS CAST CONCRETE BLOCK FACES; EXPANSION JOINTS, DIVIDING DAM INTO 85-FOOT SECTIONS; VERTICAL DRAINAGE WELL AND INSPECTION GALLERY AT TOP.

tions. A large part of the water for the other boroughs also is pumped, and much of it is pumped again at private expense to reach the upper stories of tall buildings, by pumping equipments located in the buildings. The total daily consumption of the greater city is about 500 000 000 gal., equivalent to nearly 100 gal. for each person of the census population, for public, private, and industrial purposes.

Having outgrown these supplies, the City in 1906 began the construction of a great system of new works to obtain a large additional supply from the Catskill Mountains, about 100 miles north of the City on the west bank of the Hudson. Here four contiguous water-sheds, aggregating nearly 1 000 sq. miles, are to be developed successively as needed, during the next 25 to 50 years, at a total cost estimated at $200 000 000. In Esopus water-shed, Ashokan Reservoir, of 130 000 000 000 gal. capacity, is now approaching completion; it is 12 miles long, 1 mile wide, with high-water level 590 ft. above tide, maximum water depth 190 ft., and 13 sq. miles of water surface. To impound water in this reservoir, 6 miles of dams and dikes are being built; the largest is Olive Bridge Dam, 4 650 ft. long on top, with a masonry portion 1 000 ft. long and 240 ft. high, the remainder being earth dike with concrete core-wall. The reservoir has a waste-weir of concrete 1 000 ft. long, and a dividing weir and dike 2 000 ft. long severing the reservoir into two basins.

From this reservoir Catskill Aqueduct extends 127 miles to Silver Lake Terminal Reservoir, on Staten Island, having a capacity of 400 000 000 gal., with a branch into Queensboro. Seven standard types of conduit are used in various parts of the aqueduct, depending on topographical and geological conditions, namely, cut-and-cover, built of concrete in a trench and covered with embankment; grade tunnel, through hills and mountains, at hydraulic gradient; pressure tunnel, beneath wide deep valleys, Bronx and Manhattan Boroughs and part of Brooklyn; 9½- and 11-ft. riveted steel pipes, jacketed with concrete and lined with mortar, for crossing narrow deep valleys; reinforced concrete pipes, 9½ and 17 ft. in diameter, for crossing shallow depressions and for a by-pass around Kensico Reservoir; lock-bar and riveted steel and cast-iron pipes, for delivery mains to the street distribution pipes, and flexible-jointed cast-iron pipes for crossing under The Narrows to Staten Island. The aqueduct will have a sure daily capacity of 500 000 000 gal.

About 25 miles north of the City, Kensico Distribution and Storage Reservoir, having a capacity of 40 000 000 000 gal., will contain sufficient water to supply the City for a number of weeks should the flow in the aqueduct north of it be interrupted. Kensico Dam, requiring 1 000 000 cu. yd. of masonry, will be 1 843 ft. long, and 300 ft. high. Its foundations are now being excavated to a maximum depth of 130 ft. Near both Olive Bridge and Kensico Dams, fountains, containing about 1 500 water jets each, with concrete basins 450 ft. long and 250 ft. wide, will aerate the water through 2-in. nozzles as it is drawn into the aqueduct. At several places large Venturi meters, built into the aqueduct with bronze throats, 90 in. in diameter, will measure the water. Provision has been made for a filter plant of 600 000 000 gal. maximum daily capacity when needed, by securing a site contiguous to the aqueduct 2¼ miles down stream from Kensico Reservoir.

A feature of the aqueduct which has attracted popular interest is the crossing of the Hudson, about 60 miles north of the City, 4 miles above West Point. This is accomplished by a pressure tunnel in the granite, 1 100 ft. below the river surface and 3 000 ft. long between shafts. Some of the preliminary explorations were made by pairs of inclined diamond-drill borings in the rock from 1 500 to 2 000 ft. long, driven from chambers in the shafts 250 ft. underground. The shaft at the easterly margin of the river is to serve not only as a waterway but also as an access and drainage shaft. Like the tunnel, it is 14 ft. in diameter. Its top is about 400 ft. below hydraulic gradient; nevertheless, it must be closed in such a way that it can be readily opened whenever necessary. For this cover a dome-shaped steel casting, weighing about 45 tons, was adopted, and is now being made. It is to be 17 ft. in diameter and 6 ft. 4 in. high. To hold it securely in place against the 5 000 000 lb. total upward pressure of the water, it will be anchored to the concrete shaft lining and the surrounding granite by 36 bolts, 4½ in. in diameter and 50 ft. long. Each bolt will be forged in one piece, of nickel-chrome steel, heat treated. The pull on each bolt caused by the water pressure alone will be 75 tons, to which an allowance for temperature stresses has been added.

A few incidentals of the Catskill system indicate the magnitude of the project. To replace roads to be submerged, 40 miles of new highways are being built around Ashokan Reservoir alone. As the cost of these roads, like that of other parts of the work, is defrayed

FIG. 98.—CATSKILL WATER SUPPLY: ASHOKAN RESERVOIR, WEST BASIN. OLIVE BRIDGE DAM WITH CENTRAL MASONRY PORTION COMPLETED AND EARTH EMBANKMENT AT EACH END IN PROGRESS, SHOWING CONCRETE CORE-WALLS.

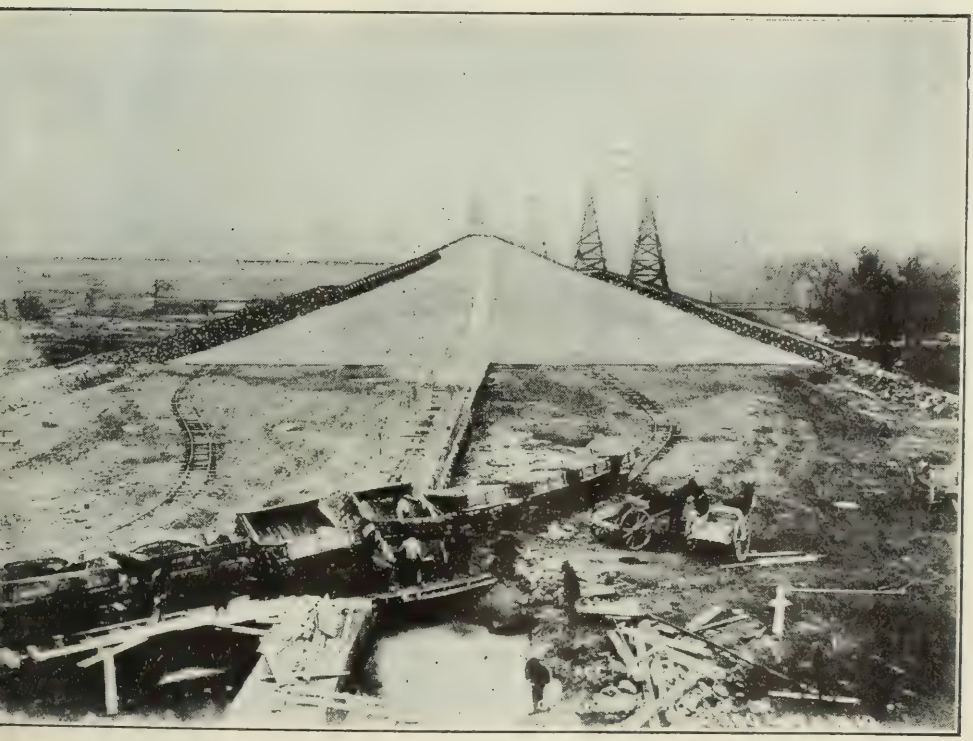


FIG. 99.—OLIVE BRIDGE DAM, EAST PORTION. MAXIMUM HEIGHT OF EARTH PORTION, 110 FEET. CONCRETE CORE-WALL. COMPLETED SECTION OF DAM DRAWN ON PHOTOGRAPH. TRESTLE TOWERS IN BACKGROUND BELONG TO THE FOUR CABLEWAYS USED IN CONSTRUCTING THE MASONRY PORTION.

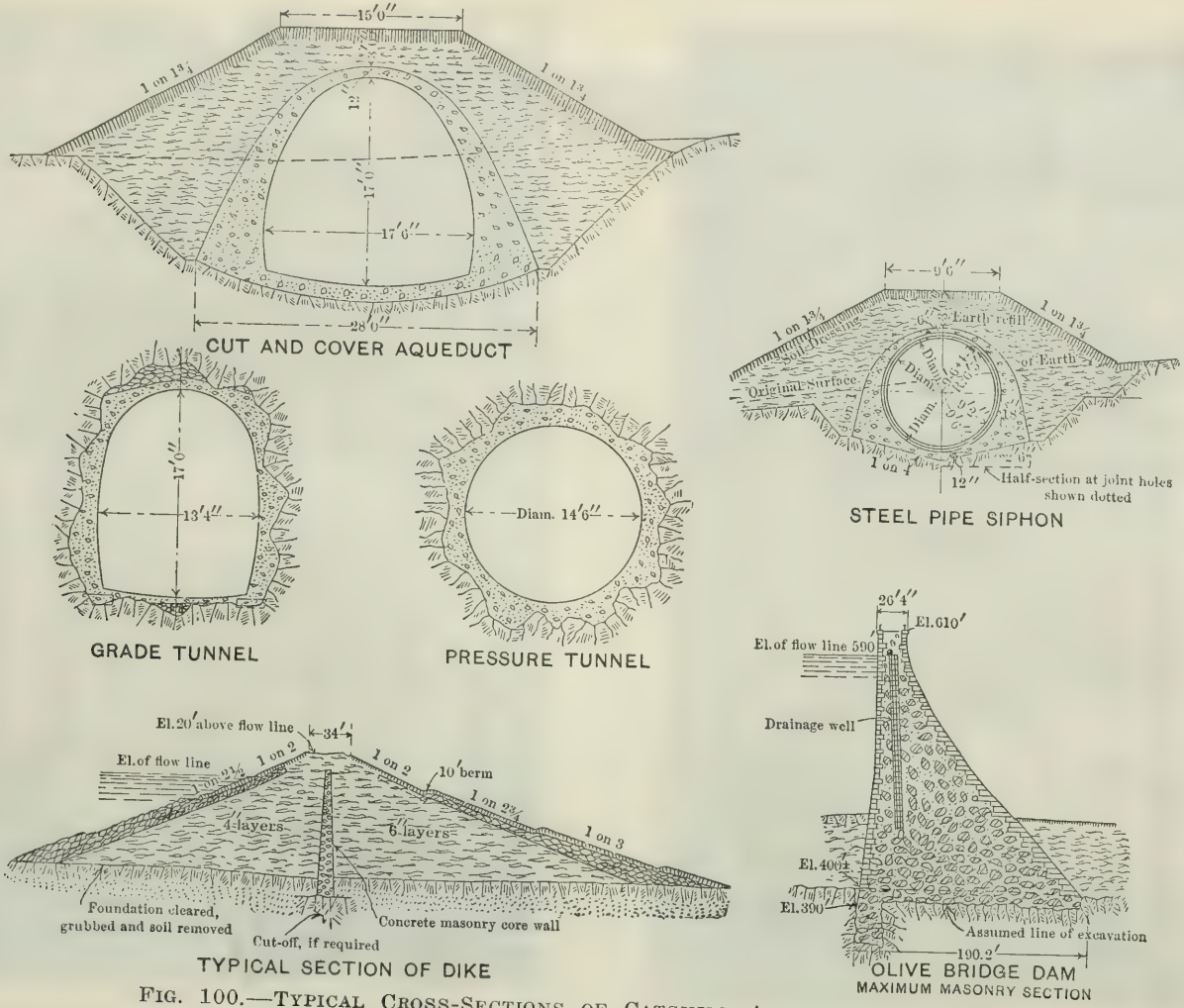


FIG. 100.—TYPICAL CROSS-SECTIONS OF CATSKILL AQUEDUCT AND DAMS.

by the proceeds of 50-year bonds, an investigation was made to determine the most permanent type of pavement. Vitrified brick paving blocks with Portland cement grout joints, on a concrete base, with concrete edgings, was chosen. Nevertheless, because of the high first cost of brick roads in this locality, and because the question as to who should

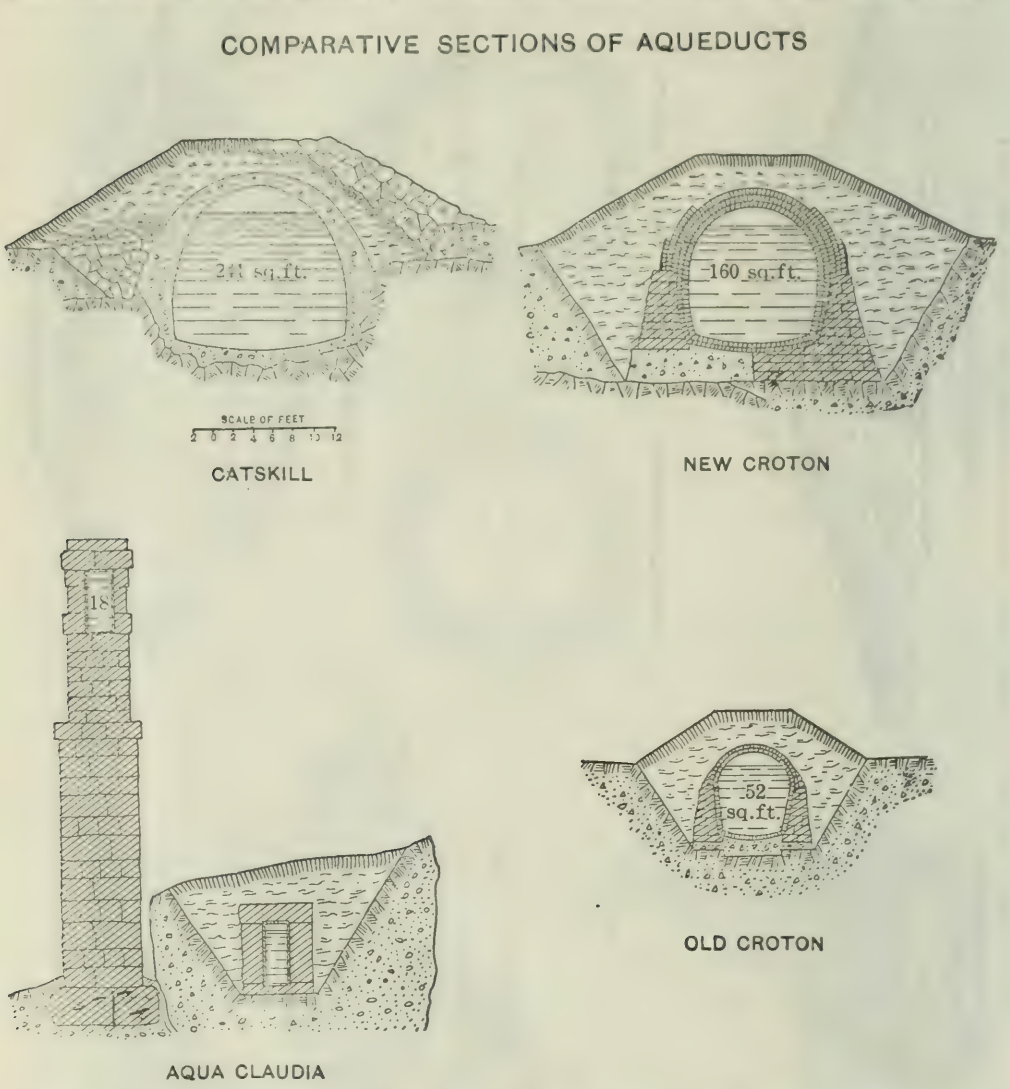


FIG. 101.—COMPARATIVE CROSS-SECTIONS OF NEW YORK AND ROMAN AQUEDUCTS.

maintain these roads had not been settled, the City authorities have decided to pave a large portion of the roads with macadam of a bituminous nature. All bridges are reinforced concrete. The City's lands along the aqueduct and around the reservoir are bounded by 100 miles of stone walls and 165 miles of heavy galvanized wire fence with reinforced concrete posts.

1797

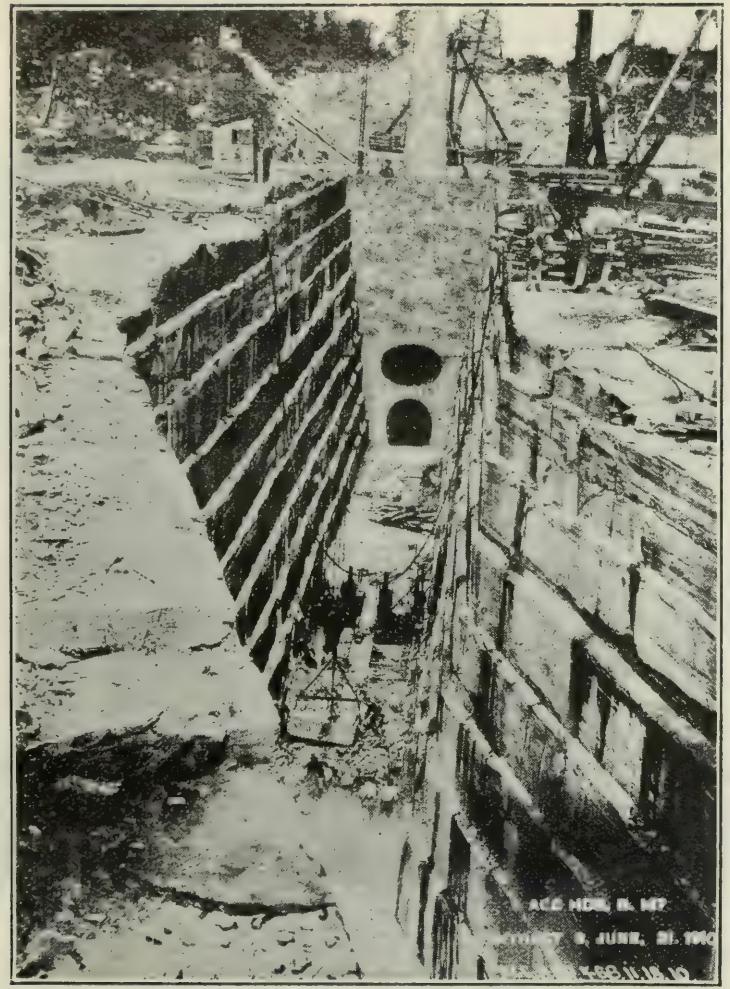


FIG. 102.—CATSKILL AQUEDUCT: DEEP TRENCH CUT IN BLUESTONE BY CHANNELING MACHINES. BEGINNING OF AQUEDUCT AT DIVIDING WEIR, DIKE OF ASHOKAN RESERVOIR.

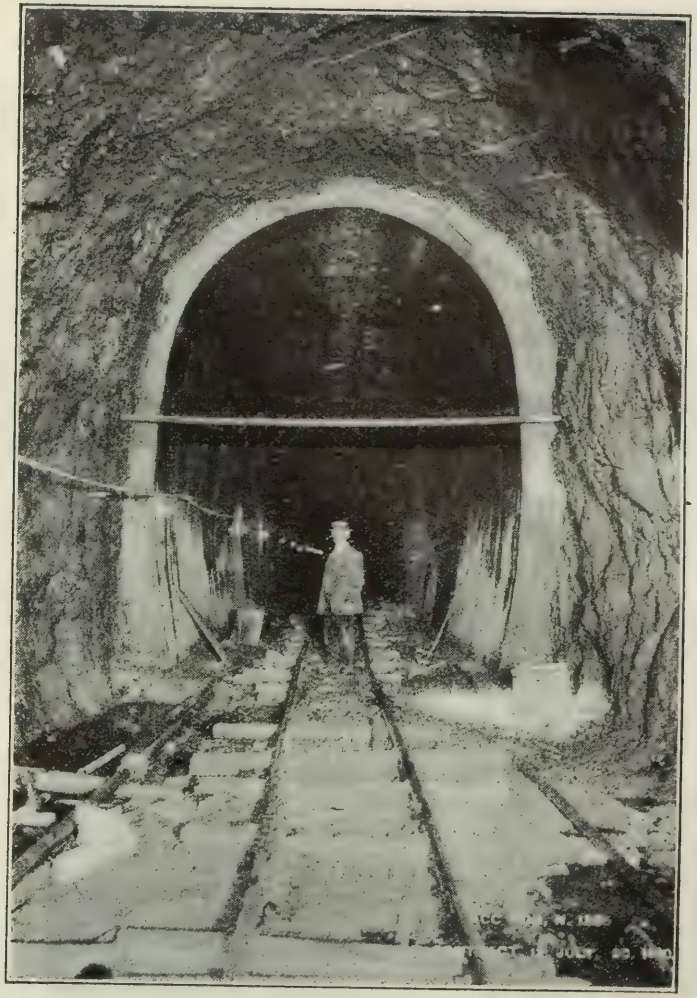


FIG. 103.—CATSKILL AQUEDUCT: A GRADE TUNNEL IN ROCK, WITH A COMPLETED SECTION OF CON-

1799

FIG. 104.—CATSKILL AQUEDUCT: CONSTRUCTION OF CUT-AND-COVER TYPE. IN THE FOREGROUND THE CONCRETE INVERT, OR BOTTOM, HAS BEEN PLACED; NEXT COME STEEL FORMS FOR INSIDE OF AQUEDUCT; IN THE MIDDLE OF THE VIEW THE OUTSIDE STEEL FORMS ALSO ARE IN PLACE, AND CONCRETE IS BEING DEPOSITED BY LOCOMOTIVE DERRICK, FROM CARS. IN THE BACKGROUND EARTH IS BEING REPLACED OVER THE COMPLETED MASONRY.

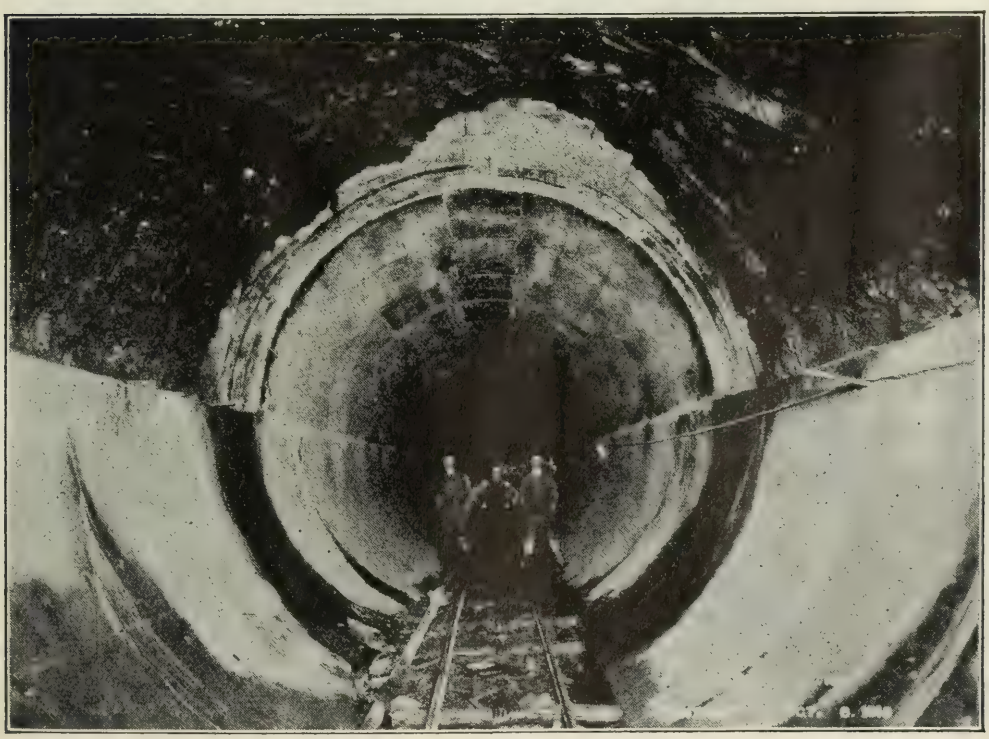


FIG. 105.—CATSKILL AQUEDUCT: A PRESSURE TUNNEL, WITH A COMPLETED SECTION OF CONCRETE LINING; 14½ FEET INSIDE DIAMETER.

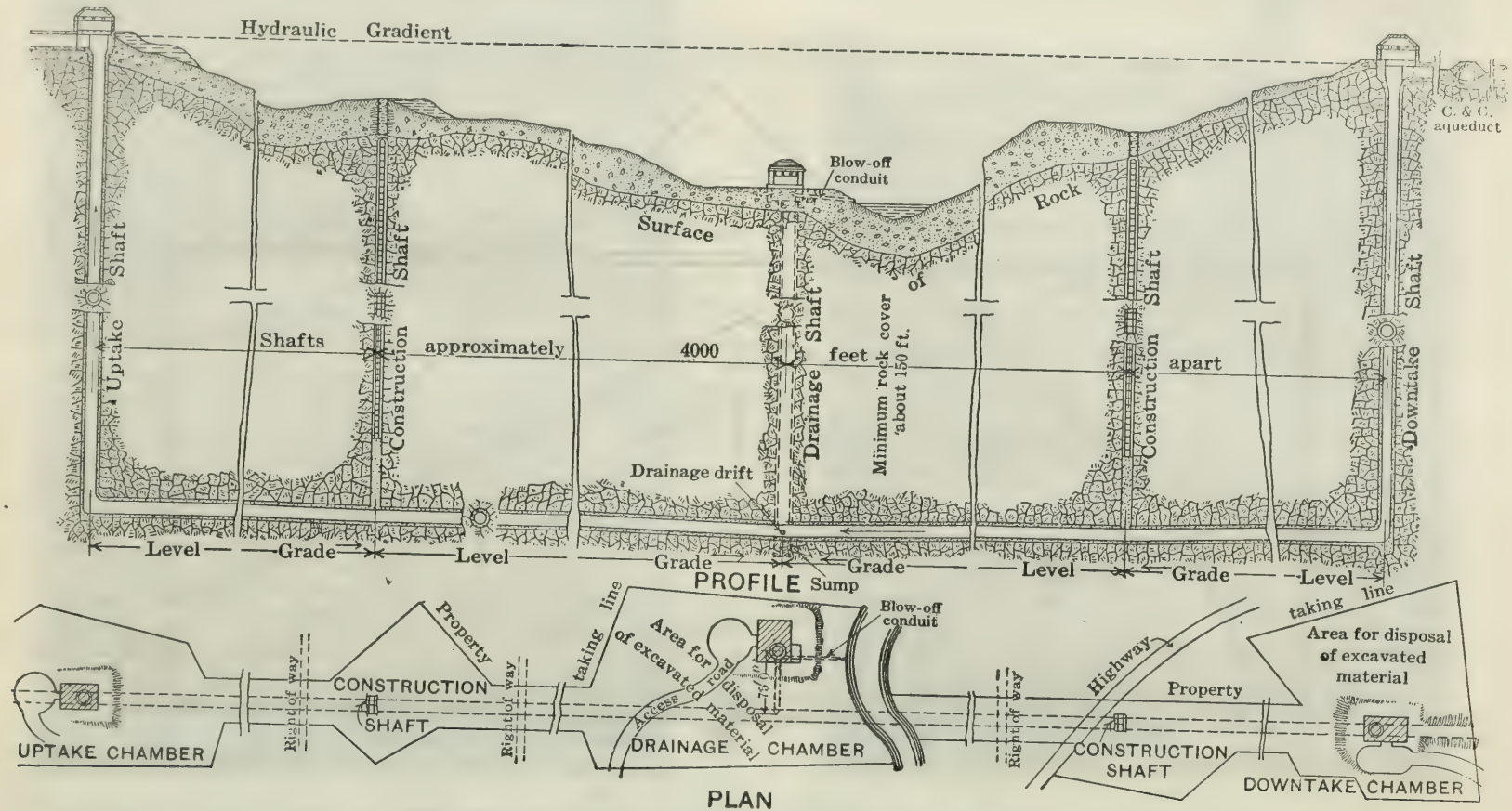


FIG. 106.—CATSKILL AQUEDUCT: TYPICAL PRESSURE TUNNEL.

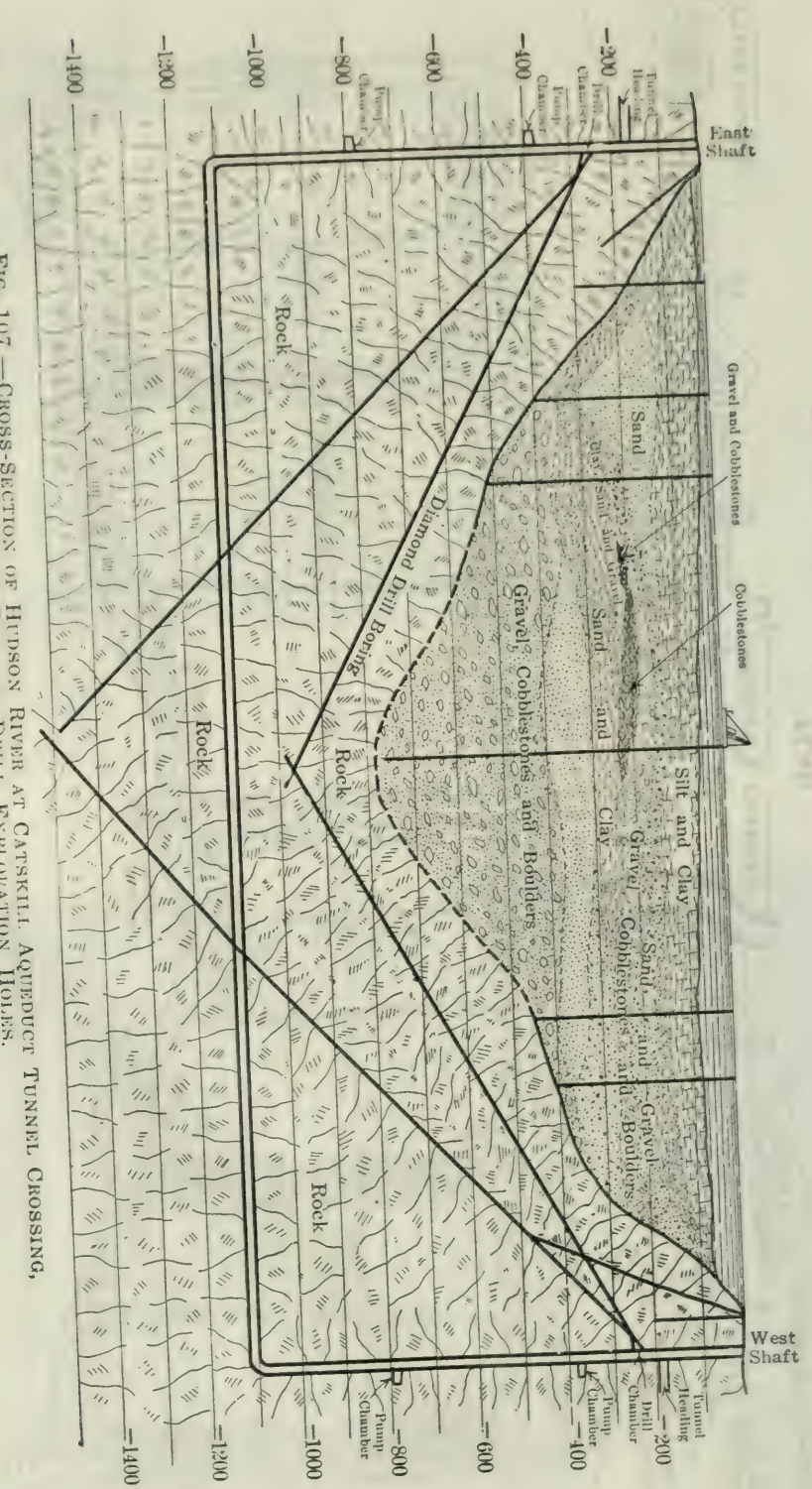


FIG. 107.—CROSS-SECTION OF HUDSON RIVER AT CATSKILL AQUEDUCT TUNNEL CROSSING, SHOWING DIAMOND DRILL EXPLORATION HOLES.

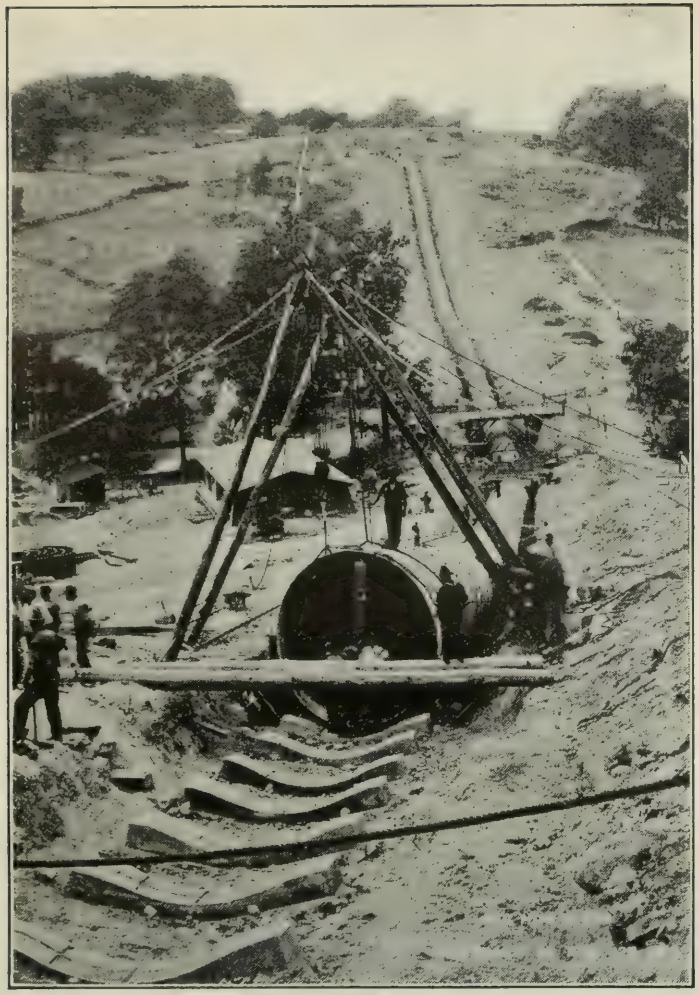


FIG. 108.—CATSKILL AQUEDUCT: HUNTER'S BROOK STEEL PIPE SIPHON; LAYING 9 FOOT 9-INCH PIPE ON CONCRETE CRADLE BLOCKS; PIPE SUBSEQUENTLY JACKETED WITH CONCRETE AND LINED WITH PORTLAND CEMENT MORTAR.

FIG. 109.—CATSKILL AQUEDUCT: KENSICO BY-PASS, FOR CONVEYING WATER AROUND KENSICO RESERVOIR; A REINFORCED CONCRETE PIPE, 12 000 FEET LONG, 11 FEET INSIDE DIAMETER; MAXIMUM HEAD, 40 FEET. CONCRETE BOTTOM AND CRADLE BLOCKS FOR SUPPORTING STEEL RINGS WHILE PLAC-

1804

Two aqueducts bring water from the New Croton Reservoir: One is 34 miles long, and has a daily capacity of 80 000 000 gal. It is mostly cut-and-cover work, and was built in 1842. The other, completed in 1891, is 31 miles long, is practically all in tunnel, and has a daily capacity of 300 000 000 gal. Plans for filtering the Croton water have been perfected, a plant having a maximum daily capacity of 400 000 000 gal. is to be built soon at Jerome Park Reservoir, in The Bronx. This rapid sand filtration plant will be the largest of the type in the world. Since the removal of color and odor is an important function in treating Croton water, coagulant is to be used. In the plant there will be a house for preparing the chemicals, a mixing chamber, six covered settling basins, of a combined capacity of 88 000 000 gal., extending over 12 acres, five gate-chambers, 80 covered mechanical filters having together a net filtering surface of 2.7 acres, and a 350 000 000-gal. filtered-water reservoir, with groined arch roof, occupying 55 acres. The filtering medium is specified to comprise not less than 8 in. of gravel in four layers, and 2½ ft. of sand having an effective size between 0.60 and 0.70 mm., with a uniformity coefficient not greater than 1.65. The rate of filtration is to be 125 000 000 gal. per acre daily. About 800 000 cu. yd. of excavation are required, and 350 000 cu. yd. of concrete masonry.

City Water Tunnel.

To deliver the great quantity of water obtained from the Catskill Mountain water-sheds into the distribution pipe systems of the City, advantage has been taken of the favorable geological conditions. From Hill View Equalizing Reservoir, at the northerly city boundary, a pressure tunnel 18 miles long is being driven in a generally southerly direction through The Bronx and Manhattan into the heart of Brooklyn. By successive steps, the diameter of the tunnel is reduced from 15 ft. for the first 40 760 ft. to 14 ft. for the next 26 190 ft., to 13 ft. for 4 540 ft., to 12 ft. for 9 050 ft., and finally to 11 ft. for the remaining 15 090 ft. It is being driven in solid ledge rock, at depths below the earth's surface ranging from 200 to 750 ft., with a rock roof nowhere less than 150 ft. thick.

For constructing the tunnel twenty-four shafts have been sunk, the most southerly seven requiring pneumatic caissons of heavily reinforced concrete to reach the bed-rock, which is overlaid by porous, water-

bearing material. Beneath the Harlem River the tunnel is 15 ft. in diameter and 330 ft. below the water surface. It crosses the East River, near the Manhattan Bridge, at a depth of 700 ft., with a diameter of 11 ft. All but three of the shafts are located in public parks or other public property, and substantially all the tunnel's length is beneath streets and parks, thus avoiding the expensive acquisition of real estate. To have delivered the Catskill water through pipes would have required thirty-two 48-in. cast-iron pipes or sixteen 66-in. steel pipes. With most of the north-south avenues already overcrowded with subsurface structures or needed for rapid transit subways, it would not have been feasible to find suitable locations for so many large water pipes; but even if room for them could have been found, their cost would have been nearly twice as great; they could not have been permanent, and the annoyances caused by laying them in the busy streets would have been unbearable.

With two exceptions, the shafts, besides their use for construction, will be the connections between the tunnel and the street mains. In each connection shaft, from the chamber at its top to a point at least 95 ft. below the top of sound rock, one or two steel pipes, some 57 and others 81 in. in diameter, are to be embedded in, and lined with, concrete; their finished waterway diameters will be 48 and 72 in. At the top of each of these riser pipes there will be a bronze pipe cross, or shaft cap, with two horizontal, flanged openings, 30, 36, or 48 in. in diameter, for the pipe connections. To each of these will be bolted an all-bronze gate-valve, ordinarily to be kept open, but closed in case of injury or repairs to the valves beyond. For routine control operations, a heavy, cast-iron, bronze-mounted gate-valve will be attached close to the bronze valve. In the top of each shaft cap an opening of the same diameter as the riser is to be made to give access to the tunnel, whenever unwatered, for inspection or repairs. A Venturi water meter in each pipe connection, outside of the chamber but near it, will measure the water delivered at that point. Automatic pressure regulating valves are provided for many of the connections, where the gravity pressure of the Catskill supply will be greater than the pressure desired in the street pipes.

As an ultimate means for closing any riser so that the bronze valves or shaft cap could be removed without putting the tunnel out of service, an all-bronze valve of special design is to be placed at the lower end

1867

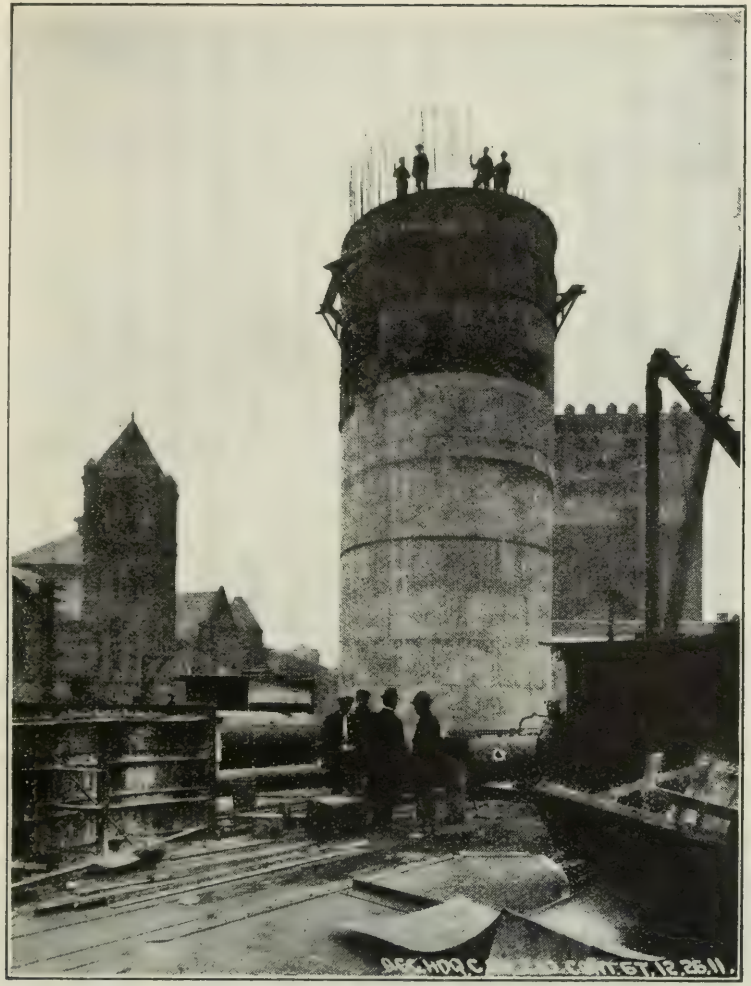


FIG. 110.—REINFORCED CONCRETE CAISSON OF ONE OF THE SHAFTS OF THE CITY WATER TUNNEL OF THE CATSKILL AQUEDUCT, IN BROOKLYN. VIEW DURING CONSTRUC-

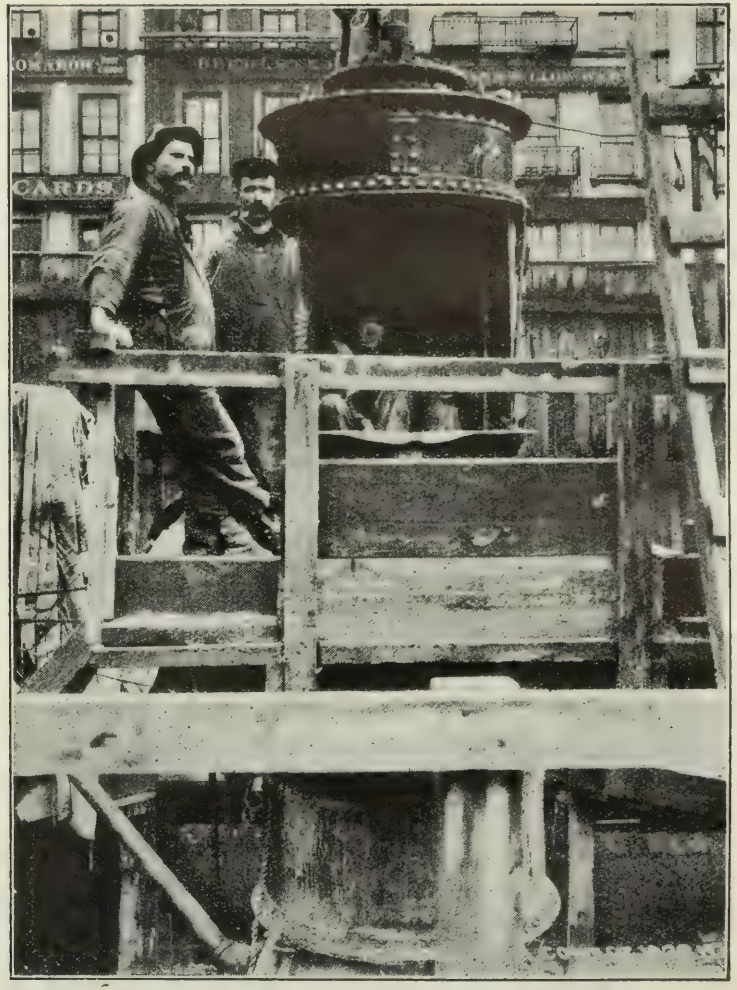


FIG. 111.—AIR LOCK ON TOP OF CONCRETE CAISSON OF ONE OF THE SHAFTS OF THE CITY WATER TUNNEL OF THE CATSKILL AQUEDUCT, IN BROOKLYN.

of the riser. As a safeguard against great loss in the event of an accident, such as a serious break in one of the large water mains, or an explosion, which might injure the top of a shaft so as to permit a great rush of water, these riser valves are to be constructed so as to close automatically whenever the velocity of water flowing through them exceeds materially a certain predetermined value. Their design provides for ready removal from the shaft, and they will be shaped so as to cause minimum hydraulic losses when wide open, which will be their ordinary position. To prevent water ram so violent as to be dangerous to the street pipes and the shaft tops, the automatic closing of the valve will be controlled by an internal dash-pot, so that no riser valve can close in less than about 10 min.

Rapid progress is being made in excavating this City water tunnel. In two years about 3 000 000 lb. of dynamite have been used in blasting out the solid rock. It is to be lined throughout with concrete. The distances between shafts range from 2 500 to 5 000 ft. Two shafts will be suitably constructed for pumping out or unwatering the tunnel. At two places great bronze valves (66 in. in diameter) will be placed across the tunnel, so that a portion of it, instead of the whole, can be put out of service for cleaning or repair. This tunnel is so unusual a feature of a water-works system that it has been described at some length. Besides the advantages already mentioned or suggested, this tunnel with its pipe line extensions will interconnect all the boroughs. The existing systems are for the most part independent of one another.

Records show that periods of greatest excess or deficiency of available quantity of water do not occur coincidentally on the three principal groups of water-sheds (Long Island, Croton, and Catskill) from which the City obtains its water supply. Consequently, another advantage arises from interconnecting these systems by the City water tunnel, which may prove of great value in the future, namely, that each system can be depended on for a somewhat greater supply than would be permissible if each were limited to its absolutely safe capacity standing alone. In other words, in the event of an extreme drought on any one water-shed, the others will be able to make up, at least in part, for the deficiency.

High-Pressure Fire System.

For better protection against fire in the most valuable business portions of Manhattan and Brooklyn, high-pressure fire-service systems

of water mains and pumping stations have been constructed and are being extended. These mains are from 6 to 24 in. in diameter, of cast iron, extra heavy, and strong enough for service pressures up to 300 lb. per sq. in. All the hydrants are of special design, and the branch pipes connecting them with the main pipes are 8 in. in diameter. In Manhattan there are now 106 miles of pipes and 18 miles are under construction; in Brooklyn there are 40 miles. Both suction and delivery piping are arranged in loop systems, thus avoiding dead ends, and interconnecting the pumping stations so that each station in a system can supply all hydrants and draw from all sources of supply.

Five pumping stations, constantly manned, are ready on a moment's notice to force water into the pipes in large quantities. Water can be drawn by these pumps from either the city's water supply system or from the harbor, the latter to be used only in emergency. Manhattan has two stations, costing about $240 000 each, exclusive of land, one on the North River front near Gansevoort Market (not far from 13th Street), and the other on the East River shore at the Brooklyn Bridge. Brooklyn has two stations somewhat smaller than those in Manhattan, one on St. Edwards Street near Fort Greene Park and the other on Joralemon Street near the East River. There is also a small station at Coney Island, which can pump 4 500 gal. per min. against 150 lb. pressure.

Each Manhattan station has six pumping units, of an individual capacity somewhat in excess of 3 000 gal. per min., with a pressure of 300 lb. per sq. in. On test, each station has pumped more than 23 000 gal. per min. using the five units which constituted the original installation. Each unit consists of a five-stage centrifugal pump and an electric motor of the induction type. All parts are interchangeable. Each motor is of approximately 1 000 h.p. Electric current for them is obtained from the New York Edison Company, which has six interconnected supply stations, with an aggregate power generating capacity of about 210 000 kw., in Manhattan, and feeders connecting with the Brooklyn Edison Company. In addition, the pumping stations are connected electrically to eighteen sub-stations of the Edison Company, equipped with converters and storage batteries, which constitute a great power reserve. Therefore it is extremely improbable that the high-pressure fire systems will ever be without power or unable to supply water whenever demand is made.

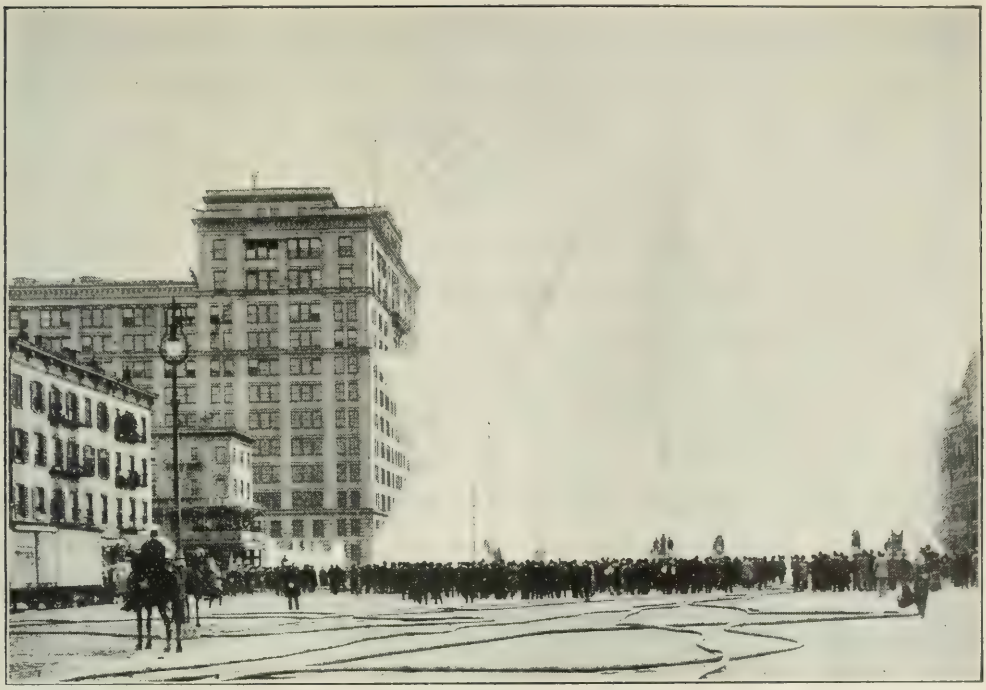


FIG. 112.—TEST OF HIGH-PRESSURE FIRE SERVICE. FIFTEEN STREAMS ARE PLAYING FROM THE STREET SURFACE. THERE ARE ALSO STREAMS FROM THE ROOF OF THE FIFTEEN-STORY BUILDING, FED BY CONNECTING TO STAND-PIPES IN THE BUILDING.

FIG. 113.—HIGH-PRESSURE FIRE SERVICE AT A WINTER FIRE, NOVEMBER 4TH, 1910. WORKING UNDER DIFFICULTIES.

Construction Camps.

In creating its stupendous modern physical structures and mechanisms, the metropolis has not been wholly unmindful of the humanitarian aspects of its engineering activities. Not only have "labor laws" been enacted by the State Legislature in greater or lesser wisdom, but into many contracts engineers and executive officers have written requirements for the better housing of the men, the more complete safeguarding of life and limb, and the sanitary regulation of construction camps, not alone for the benefit of the dwellers therein, but also for the greater protection of the communities affected directly and indirectly.

In constructing the works for its new Catskill water supply, the City has set a high standard in several particulars. Sanitarians, physicians, and surgeons have been engaged to assist the engineers and contractors in caring for the health of the men and guarding from pollution all public and private water supplies. In each of the numerous principal camps have been housed from a few hundred to several thousand human beings, besides hundreds of draft animals. Stables for the animals have been kept at a distance from the quarters for the men. In social classification, occupants of the camps have ranged from common laborers to superintendents and engineers. Many men, of all ranks, have been accompanied by their families. For the latter, separate cottages have been built. In general, the City has provided land for the camps and has insisted that they be built on well-drained, healthful sites, with ample spaces between houses and liberal allowance of air and light for each occupant. All camps have had provisions for sewage and garbage disposal, depending on the size and location. Commonly, garbage and excreta have been incinerated. Ample water supplies of good quality, and fire-fighting facilities, have been required. Physical examinations of employees and applicants have been made in order to detect communicable diseases. Food supplies have been inspected. All large camps have had hospitals, with surgeons and nurses in attendance. Camp City, at Ashokan Reservoir, although temporary, is a completely equipped modern community, having, in addition to the public utilities already alluded to, day and evening schools, a bank, post-office, general store, bakery, streets, electric lights, telephones, churches, Young Men's Christian Association, and an amateur band of music.

As the Catskill Aqueduct, for nearly 10 miles, is located on the Croton catchment area, and crosses the principal reservoir in a deep tunnel, with a shaft on either shore, about 2 000 ft. from the intakes to the two Croton Aqueducts, special sanitary precautions were taken. All camps were surrounded by man-proof fences and illuminated by night, for better control. Filters and chemical treatment were provided for all sewage and for drainage from camp areas, tunnels, and spoil banks; incinerators, of types similar to those devised originally for the United States army, were provided for defecation in the camps, and small shanties with water-tight removable receptacles were placed at convenient points on the works. Garbage was separately incinerated. That these precautions were effective is proven by the prevailing good health in the camps and the absence of epidemics of water-borne diseases in the City and the other communities within the water-sheds of which the operations have been situated. That labor forces, reaching a maximum total of 5 000 men, could have been resident for three years or more in camps on the most important catchment area of the City's water supply, and there employed in heavy excavation and masonry construction, without once causing sanitary troubles, is an achievement worthy of more than passing reference.

The total capital expenditure for water-works has been about $300 000 000, including the portion of the Catskill system now in hand.

Conclusion.

In this relatively young country, still only partly developed, and with many of its earlier constructions of temporary character, its engineers and architects, in all the branches of these constructive professions, have constantly great tasks to perform. Nowhere are more works of noteworthy magnitude concentrated than within 100 miles from New York City as a center. It is no exaggeration to state that, at all times during the past few years, private, public, and semi-public undertakings, aggregating $1 000 000 000, have been in various stages of progress within this area. The promise for the immediate future is for a continuance of these achievements and activities. The forces which have made New York continue in operation, and new forces are contributing to the City's further expansion. All these engineering achievements and activities of this great young City grew from the humble beginning at the southerly tip of Manhattan Island,

Fort New Amsterdam. Most of the modern greatness of the Metropolis has been created within the memory of men not yet 40 years old, the parents of some of whom can well remember carrying lanterns when going to "evening meeting" up to 14th Street, and others of whom once pastured their cows where Central Park now is, whose homes and churches have been supplanted by many-storied towering hives of commerce and industry.

Acknowledgments.

Like the engineering achievements of New York City, this paper is the result of co-operation, and it gives the compiler pleasure to acknowledge his indebtedness. Much of the material was collected by Robert Ridgway, Frederick C. Noble, Arthur S. Tuttle, Members, Am. Soc. C. E., and Julian P. W. Richmond, Assoc. M. Am. Soc. C. E. Information and views were generously contributed by many of the city departments and their officials; by several public utility corporations; by Alfred Noble, Past-President, Am. Soc. C. E., Chief Engineer of the East River tunnel work for the Pennsylvania Railroad; George A. Harwood, M. Am. Soc. C. E., Chief Engineer of the Electric Zone Improvements of the New York Central and Hudson River Railroad Company; C. M. Jacobs and J. V. Davies, Members, Am. Soc. C. E., Engineers for the North River tunnels of the Pennsylvania Railroad, and the Hudson and Manhattan tunnels; George H. Pegram, M. Am. Soc. C. E., Chief Engineer, Interborough Rapid Transit Company; Alfred Craven, M. Am. Soc. C. E., Chief Engineer, and Mr. Louis D. Fouquet, Engineer, Sewer Division, Public Service Commission; Alexander Johnson, Assoc. M. Am. Soc. C. E., Chief Engineer, Bridge Department; J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer, Board of Water Supply; George A. Soper, M. Am. Soc. C. E., President, and Kenneth Allen, M. Am. Soc. C. E., Chief Engineer, Metropolitan Sewerage Commission; I. M. de Varona, M. Am. Soc. C. E., Chief Engineer, and William W. Brush, M. Am. Soc. C. E., Deputy Chief Engineer, Department of Water Supply, Gas, and Electricity; George W. Tillson, M. Am. Soc. C. E., Consulting Engineer, Borough of Brooklyn; Edwin J. Fort, M. Am. Soc. C. E., Chief Engineer, Bureau of Sewers, Brooklyn; B. F. Cresson, Jr., M. Am. Soc. C. E., First Deputy Commissioner, and S. W. Hoag, Jr., M. Am. Soc. C. E., Deputy Chief Engineer, Department of Docks and Ferries;

Mr. Charles B. Stover, President, Department of Parks; Ernest P. Goodrich, M. Am. Soc. C. E., Consulting Engineer, and Charles E. Gregory, Assoc. M. Am. Soc. C. E., Deputy Consulting Engineer, Manhattan; R. H. Gillespie, M. Am. Soc. C. E., Chief Engineer, Office of President of Borough of The Bronx; T. S. Oxholm, M. Am. Soc. C. E., Engineer, Office of President of Borough of Richmond; Amos Schaeffer, M. Am. Soc. C. E., Consulting Engineer, Office of President of The Bronx; Mr. C. C. Simpson, Consolidated Gas Company; George A. Orrok, M. Am. Soc. C. E., Mechanical Engineer, New York Edison Company; Mr. C. Foster, Bush Terminal Company; Gustav Lindenthal, M. Am. Soc. C. E.; Toch Brothers; Mr. Charles P. Berkey, Expert Geologist; Mr. F. E. Congdon, New York Telephone and Telegraph Company, and many others.

1817

t' Fort nieuw Amsterdam op de Manhatans
FORT NEW AMSTERDAM (NEW YORK). 1651.

FIG. 114.

1819

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1275

THE ELEVATION OF THE TRACKS OF THE PHILADELPHIA GERMANTOWN AND NORRISTOWN RAILROAD, PHILADELPHIA, PA.*

By Samuel Tobias Wagner, M. Am. Soc. C. E.

With Discussion by Messrs. J. P. Snow, E. W. Lewis, Charles Rufus Harte, George S. Webster, and Samuel Tobias Wagner.

It is the purpose of this paper to present the more interesting features of the work of abolishing grade crossings on the line of the Philadelphia, Germantown, and Norristown Railroad, a leased line of the Philadelphia and Reading Railway, in the City of Philadelphia, between Green Street and the Richmond Branch Crossing near Wayne Junction.

The Philadelphia and Reading Railway enters the heart of the City of Philadelphia, to the Reading Terminal at Twelfth and Market Streets, by two principal routes:

First.—From the main line on the Schuylkill River through the Subway on Pennsylvania Avenue.

Second.—From Wayne Junction *via* private right of way and Ninth Street.

The avoidance of grade crossings on the first line has been described in the *Transactions*.† It is the purpose of this paper to describe the second, which has recently been completed.

* Presented at the meeting of June 4th, 1913.

† *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 1; and Vol. XLVIII, p. 470.

Both these lines are part of the Philadelphia Division of the Philadelphia and Reading Railway.

The tracks, at the time work was started, were elevated from the Reading Terminal as far north as Green Street, where a descent was made to the street level at Fairmount Avenue. From this point to Wayne Junction, a distance of 4.3 miles, there existed in 1906 no less than 28 grade crossings, some of them being very dangerous.

The Philadelphia, Germantown, and Norristown Railroad was built in 1833, at which time the major part of the territory traversed was open fields. As the City grew and new streets were opened, new grade crossings were being constantly created, and conditions became very bad on account of very heavy and rapid train movements over the line.

Previous to 1900 the exceptionally bad crossings existing at Columbia Avenue and Broad Street were abolished, under a city ordinance of December 26th, 1890, which authorized the construction of the Reading Terminal. In both these cases the street grades were raised and the highways carried over the railroad.

Prior to 1906 a number of studies looking toward the abolishment of these grade crossings were made, both by the City and the Railway Company, all being based on the elevation of the tracks as the best and most economical method of accomplishing the desired end. The total estimated cost of the work was as follows:

Green to Broad Streets	$5 779 740
Seventeenth Street to Richmond Branch	1 880 000
Total	$7 659 740

Legislation.

Between 1903 and 1906 a number of conferences were held between the Mayor of the City and the management of the Railway Company for the purpose of reaching an agreement as to the general plan to be followed and the division of cost of the work. At first it was the intention to do away with the grade crossings only between Green and Broad Streets, but, as the work developed, the section north of Broad Street was included, by mutual consent.

After considerable discussion, a form of ordinance authorizing the Mayor to enter into an agreement with the Railway Company for the

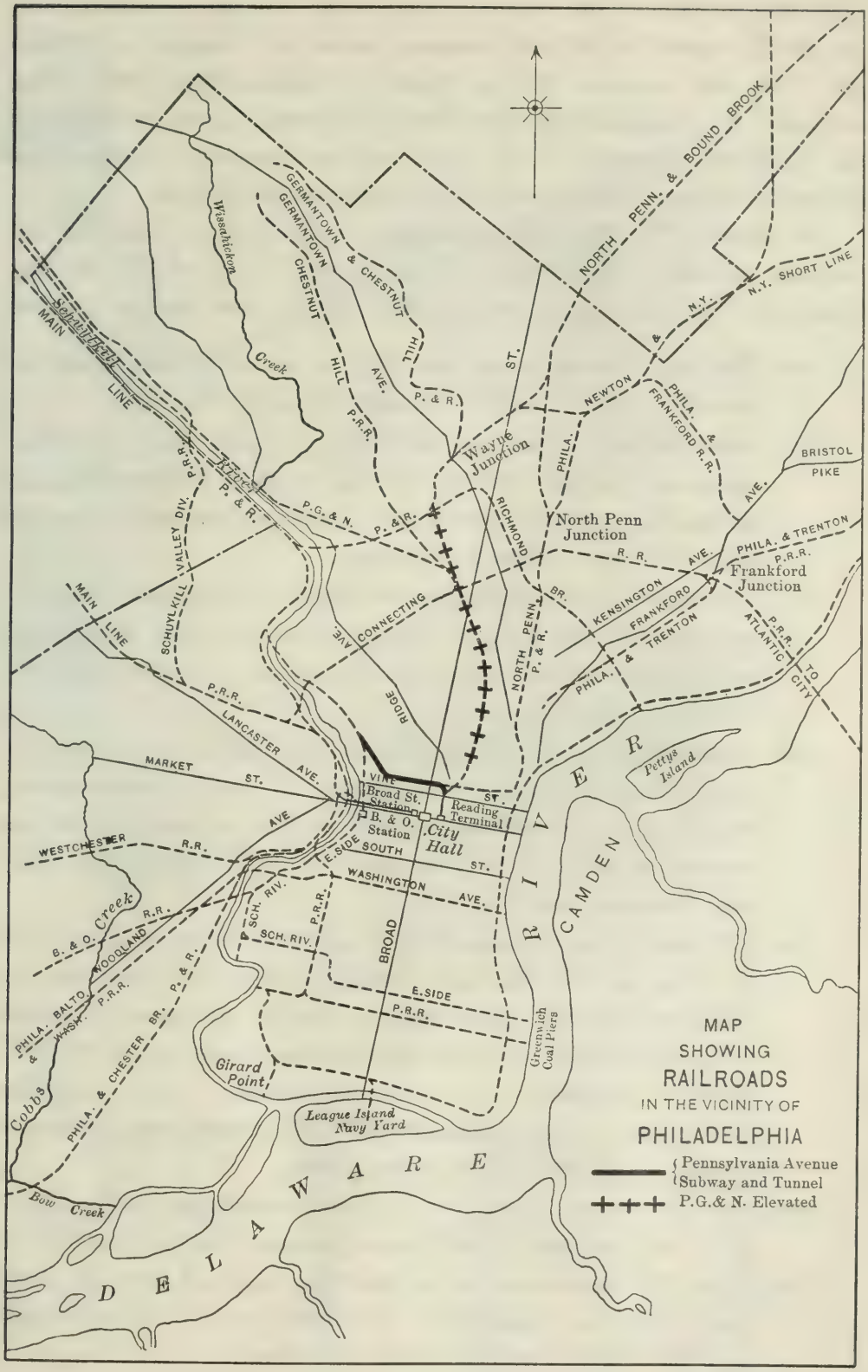

NORTH PENN. & BOUND BROOK
N.Y. SHORT LINE
N.Y.
GERMANTOWN & CHESTNUT HILL P. & R.
GERMANTOWN CHESTNUT HILL P.R.R.
Wissahickon Creek
MAIN LINE
SCHUYLKILL VALLEY DIV. P.R.R.
Wissahickon P. & R.
P.G. & N.
P. & R.
Wayne Junction
PHILA. NEWTON & FRANKFORD R.R.
BRISTOL PIKE AVE.
North Penn Junction
RICHMOND
PHILA. & TRENTON R.R.
P.R.R.
Frankford Junction
KENSINGTON AVE.
FRANKFORD BR.
NORTH PENN. P. & R.
CONNECTING AVE.
RIDGE
PHILA. & TRENTON
P.R.R. ATLANTIC TO CITY
MAIN LINE
P.R.R.
LANCASTER AVE.
MARKET ST.
VINE ST.
Broad St. Station
B. & O. Station
Reading Terminal
City Hall
SOUTH ST.
WASHINGTON AVE.
WESTCHESTER R.R.
E. SIDE
SCH. RIV.
P.R.R.
B. & O. Creek
WOODLAND
PHILA. BALTO & WASH. P.R.R.
PHILA. & CHESTER BR. P. & R.
BROAD
E. SIDE P.R.R.
Girard Point
Greenwich Coal Piers
League Island Navy Yard
Cobbs Creek
Bow Creek
Pettys Island
DELAWARE RIVER
CAMDEN
MAP
SHOWING
RAILROADS
IN THE VICINITY OF
PHILADELPHIA
Pennsylvania Avenue Subway and Tunnel
P.G.& N. Elevated

FIG. 1.

work was introduced into Councils and, after discussion in that body, was finally agreed to by both interests, passed by Councils on October 4th, 1906, and approved by the Mayor on October 13th, 1906. This ordinance also included the abolishment of certain grade crossings on the Richmond Branch of the same railway between Somerset and Richmond Streets, which work will not be described here.

On March 28th, 1907, the agreement authorized by Councils was formally signed by the proper officials of the City and the Railway Company. This agreement, among other things, provided, as follows:

1st. The legal authority for the City to revise the city plans, on account of the changes of grade of the streets, etc.

2d. Authorization of the city officials to prepare plans, specifications, and contracts for the changes of grade of the street, and the opening of new streets, said plans and contracts to be approved by the Railway Company.

3d. Authorization of the Railway Company to prepare plans and contracts for the work of raising its tracks, with the bridges and all other appurtenances, said plans and contracts to be approved by the City.

4th. Authorization of both the City and the Railway Company to employ engineers and inspectors; also to provide for emergency work in the maintenance of railroad and highway travel, not covered by contract, by force account on the part of the Railway Company and the City.

5th. Division of the cost equally between the Railway and the City as long as the Railway Company does not increase its facilities over and above what it had at the time of passage of the ordinance; all other additional expenses incurred by the Railway Company to be at its sole expense. The cost of the work to be equally divided includes all consequential damages caused by the change of street grades.

6th. For the proper disposal (and proper credit to both parties) of all old material or real estate purchased and not used.

7th. That the representatives of the City and the Railway Company shall be, respectively, the Director of the Department of Public Works and the Chief Engineer.

1874

PLAN
OF THE
PROPOSED ELEVATION OF TRACKS
FOR THE ABOLISHMENT OF ALL GRADE CROSSINGS
ALONG THE
PHILADELPHIA, GERMANTOWN & NORRISTOWN RAILROAD
FROM
GREEN ST. TO BROAD ST

SCALE OF FEET
HORIZONTAL 0 100 200 300 400
VERTICAL 0 10 20 30 40

Notes:
Limit of change of street grades
Indicated thus

PLAN SHOWING ORIGINAL CONDITIONS

PLAN SHOWING FINISHED CONDITIONS

TYPICAL SECTION OF TEMPORARY TRESTLE DURING CONSTRUCTION

TYPICAL SECTION OF MASONRY CONSTRUCTION COMPLETED

TROUGH FLOOR CONSTRUCTION ON STEEL VIADUCT

TYPICAL SECTION OF STEEL VIADUCT DURING CONSTRUCTION

TYPICAL SECTION OF STEEL VIADUCT COMPLETED

PROFILE ALONG CENTRE LINE

General Description.

The following is a general description of the work: Beginning at Green Street, to which point from the south the tracks had already been elevated, they run level at an elevation of 57.44 to a point near Brown Street. A slight hump is placed in this profile to cross the engine passageway in the Green Street Yard. As far as Brown Street, the four main tracks are carried on fill between concrete retaining walls with steel plate-girder bridges crossing the engine passageway and Fairmount Avenue at four different levels. At Fairmount Avenue the grade of the street was lowered so as to allow 14 ft. beneath the bridges leading to the engine yard on the east and west sides of the main tracks. A new locomotive coaling station, with a storage capacity of 2 000 tons, is located in the east yard.

At Brown Street the tracks are over the bed of Ninth Street and are carried on a four-track, steel viaduct with plate-girder spans, generally 50 ft. long, supported by transverse girders and three lines of columns, and, in turn, supporting a solid steel floor, which is waterproofed. This viaduct is on a grade of 0.5%, and is on a tangent as far as Master Street. At Girard Avenue a new elevated station is supported over the street.

Between Thompson and Master Streets, and Eighth and Ninth Streets, the old freight yard is raised to the new level with a ramp approach from Eighth and Master Streets.

At Jefferson Street the tracks are supported on earth fill with third-class, masonry retaining walls and concrete abutments, and this type of construction continues to the end of the line at the Richmond Branch Crossing. At Jefferson Street the line leaves Ninth Street and occupies its own right of way. This right of way, 66 ft. in width, formerly extended as far south as Girard Avenue, but, under the ordinance authorizing the work, it was released to the City for street purposes beneath the viaduct.

The 0.5% grade extends about to Oxford Street, where the tracks are level at an elevation of 74.5, to a point near Montgomery Street.

Between Oxford Street and Columbia Avenue is the Columbia Avenue Station, having island platforms with stairways leading to the main waiting- and baggage-rooms at the southwest corner of Ninth Street and Columbia Avenue.

The tracks cross Berks and Norris Streets on concrete arches, replacing old iron plate-girder bridges which formerly carried the low-level tracks over these streets.

Between Berks and Norris Streets on the west side is a freight yard raised nearly to the new level, with a ramp approach parallel to Tenth Street, and on the east side, on property of the Railway Company, there are two large commercial coal yards. At the northeast corner of Tenth and Norris Streets there is a smaller coal yard.

Between Berks and Norris Streets the tracks ascend on a 0.7% grade to the summit at York Street and then descend on a 1% grade to Broad Street, where the old level is reached.

Between Norris and Cumberland Streets there are five tracks crossing a large number of streets, at very acute angles, on steel bridges of plate-girder type with solid water-proofed floors. Between York and Cumberland Streets there are two elevated freight yards, and north of Cumberland Street there is a car-cleaning yard on the east side.

Between Broad Street and a point near Seventeenth and Indiana Streets no work was required, as there were no grade crossings in this territory.

From Seventeenth and Indiana Streets to Tioga Street, the grade is 1.06%, and there are five tracks as far as Westmoreland Street and four north of that point. In order to obtain the 1.06% grade, it was necessary to raise a truss bridge of the Philadelphia, Germantown, and Chestnut Hill Railroad (Pennsylvania System) about 4 ft. This was done at a cost of $18 384.47.

The new elevation required changes in the grades of Allegheny Avenue, Westmoreland Street, Ontario Street, and Hunting Park Avenue, and slight changes at Atlantic and Venango Streets. The changes at Allegheny Avenue and Westmoreland and Ontario Streets necessitated those in a number of other streets shown on the general plans, Plates LXVI and LXVII.

New stations were constructed at Tioga and Nicetown, and a coal yard of the Railway Company was reconstructed at Hunting Park Avenue.

Preparation of Plans and Contracts.

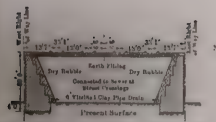
Plans and specifications for the various contracts were duly prepared for the work by the Railway Company and the City, and were

PLATE LXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1275.
WAGNER ON
TRACK ELEVATION, PHILADELPHIA, PA.

1914A.

PLAN
OF THE
PROPOSED ELEVATION OF TRACKS
FOR THE ABOLISHMENT OF ALL GRADE CROSSINGS
ALONG THE
PHILADELPHIA GERMANTOWN & NORRISTOWN RAILROAD
FROM
INDIANA ST. TO THE RICHMOND BRANCH

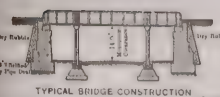
SCALE OF FEET
HOR. 0 50 100 200 300 400 500
VERT. 0 10 20 30 40 50



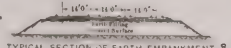
TYPICAL SECTION OF MASONRY CONSTRUCTION

NOTE.- Limit of change of street grades indicated thus

TYPICAL SECTION OF BRIDGE FLOOR CONSTRUCTION



TYPICAL BRIDGE CONSTRUCTION



TYPICAL SECTION OF EARTH EMBANKMENT

submitted for approval to the other interest before asking for bids. All work was advertised publicly, and awards were made to the lowest bidder.

Division Into Contracts.

In the summer of 1906, it was decided to divide the work into contracts so that it could be attacked from the greatest number of points at one time, and so that, as far as possible, the work requiring special skill could be separated in order to reduce the number of contracts to a minimum. All track work was done by Railway Company's forces on what were known as emergency orders.

Tables 1 and 2 show the contracts into which the work was divided. All numbers from 1 to 100 were executed by the Railway Company; those above 100 by the City.

Emergency Orders.

The ordinance provided that in case of emergency work in maintaining railroad or highway travel, and in other cases where necessary—not covered by contract—either the Railway Company or the City should have authority to employ laborers, hire machinery, and purchase materials to perform such work. Such work, when jointly agreed upon, constituted what was known as an emergency order. On the part of the Railway Company, these orders generally applied to the construction of temporary and permanent tracks, the work being done by the Operating Department; and on the part of the City to work connected with the maintenance of underground municipal structures. They were also used for keeping account of numerous minor additional betterment items desired by the Railway Company during the progress of the work, and paid for entirely by them. All labor on pay rolls was carefully checked, and all prices of materials were approved prior to purchase.

Temporary Yards.

Besides the heavy passenger business conducted over the line which was to be elevated, involving about 467 train movements in 24 hours, the Railway Company operated freight yards at Eighth and Master, Tenth and Berks, and Thirteenth and Huntingdon Streets, and a locomotive coaling station and engine repair yard at Ninth and Green Streets.

TABLE 1.—List of Contracts with Railway Company.

No	Description.	Contractor.	Date of contract.	Date of final estimate.	Amount.
1	Masonry and embankment, Green to Brown	James McGraw Co.	6- 4-09	8-31-12	$291 561.46
2*	Foundations for steel viaduct	Owen J. Evers	6-14-09	8-31-09	6 383.38
2R	Foundations for steel viaduct	James McGraw Co.	11- 6-09	9-30-10	179 538.56
3	Bridges and viaduct, Green to Jefferson	McClintic-Marshall Cons. Co.	6-15-09	12-30-11	842 741.57
4	Masonry and embankment, Jefferson to Montgomery	Armstrong & Latta Co.	7-31-09	9-30-11	171 304.96
5	Masonry and embankment, Montgomery to Susquehanna	Armstrong & Latta Co.	1-27-08	11- 6-09	460 993.79
6	Bridges, Oxford, Columbia Ave., Montgomery	S. F. Shoemaker & Co.	7-30-09	1-31-12	96 300.66
7	Masonry and embankment, Susquehanna to Broad	James McGraw Co.	1-21-08	9-30-09	290 580.77
8	Bridges, Berks to Broad	McClintic-Marshall Cons. Co.	2-15-08	8-31-10	340 794.53
9	Masonry and embankment, Indiana to Tioga	Armstrong & Latta Cons. Co.	8- 2-09	6-30-11	106 812.17
10	Masonry and embankment, Tioga to Richmond Branch	Enos L. Seeds	8-18-09	6-30-11	111 275.19
11	Bridges, Indiana to Richmond Branch	American Bridge Co.	8-17-09	11-29-11	267 559.51
12†	Masonry, etc., Master St. Yard	Owen J. Evers	6-14-09	8-31-09	16 667.12
12R	Masonry, etc., Master St. Yard	James McGraw Co.	11- 6-09	10-31-11	89 397.59
14	Masonry, etc., York St. Yards	Armstrong & Latta Co.	1-17-08	10-31-11	67 059.23
15	Columbia Ave. Station	Irwin & Leighton	4-21-10	11-30-12	217 134.15
16	Tioga Station	Enos L. Seeds	1-20-10	9-30-12	62 886.64
17	Nicetown Station	Fred. A. Havens & Co.	1-22-10	12-30-11	43 554.57
19	Signals, Green to Broad	Union Switch & Signal Co.	7- 2-09	1-31-12	127 462.90
21	Locomotive coaling station	Link Belt Co.	4-30-09	12-31-10	96 670.13
26	Temporary engine yard, Wayne Junction	Armstrong & Latta Co.	1-22-08	6-30-08	51 571.60
27	Temporary freight yard, Nineteenth and Indiana	Reilly & Riddle	10- 4-07	5-30-08	40 381.40
28	Car-cleaning yard, Huntingdon St.	Edw. Fay & Son	7-13-08	6-30-09	53 326.82
29	Engineer's office, Huntingdon St.	Edw. Fay & Son	7-12-07	12-31-07	8 920.94
30	Coal pocket yard, Ninth and Berks	Wm. Steele & Sons Co.	5-25-08	3-31-09	273 578.29
31	Coal pocket yard, Tenth and Norris	Wm. Steele & Sons Co.	10-29-08	6-30-09	42 884.33
32	Masonry, etc., freight yard, Tenth and Berks	Armstrong & Latta Co.	10-16-08	5- 7-10	70 350.03
33	Signal bridges, Broad to Norris	Steward & Stevens	7- 8-09	2-28-10	4 155.92
35	Drainage, York St. Yard	O'Toole, Walls & Dempsey	10-24-08	8-31-09	2 762.83
36	Fences and gates, Berks to Broad	W. W. Lindsay & Co.	5-13-09	4-30-10	14 029.35
38	Signal towers, Green to Norris	Thos. J. Trafford	10-18-09	2- 7-11	8 567.46
39	Store house, Ninth and Green	Geo. A. Glenn & Co.	3- 2-10	2-28-11	27 034.15
41	Engine water supply, Ninth and Green	Burd P. Evans & Co.	1-25-10	2-28-11	6 107.27
46	Coal yard, Hunting Park Ave.	Enos L. Seeds	12-23-10	6-30-11	18 884.26
47	Signal bridges, Green to Norris, Tioga Section	L. F. Shoemaker & Co.	5-24-10	11-29-11	12 301.65
48	Tool house, Tenth and Diamond	Fred. A. Havens & Co.	4-22-10	8-31-10	3 583.70
53	Platforms—Powers, Weightman, Rosengarten	Belmont Iron Works.	6-18-09	2- 4-11	5 494.00
54	Coal pockets—Powers, Weightman, Rosengarten	Burd P. Evans	2-15-11	4-29-11	1 842.99
E.O. 57	Girard Avenue Station	Irwin & Leighton	10-20-11	8-31-12	54 602.40
E.O. 85	Signal towers, Broad to Norris	Armstrong & Latta Co.	11-22-09	7-30-10	10 820.97

*Contractor defaulted—$35 627.09 recovered from sureties.

†Contractor defaulted—$10 048.31 recovered from sureties.

TABLE 2.—List of Contracts with City.

No.	Description.	Contractor.	Date of contract.	Date of final estimate.	Amount.
101	Sewers, 13th, Dauphin to Cumberland	Millard Cons. Co	8-27-07	7-27-08	$29 497.98
102	Sewers, on 16th St., and from 16th to 17th on Allegheny Ave	David McMahon	8-27-07	8-29-08	33 861.39
103	Sewers, Allegheny Ave., from 17th to 19th; Nineteenth St., Allegheny Ave. to Westmoreland	Millard Cons. Co	8-27-07	11-18-08	46 927.39
104	Sewers, Nineteenth, Westmoreland to Venango; Venango, 19th to 20th	Millard Cons. Co	8-27-07	11-18-08	71 100.92
105	Sewers, Venango, 20th to 21st; on 21st to Blabon, Hunting Park Ave	Millard Cons. Co	8-27-07	11-18-08	36 602.56
108	Street changes, Fairmount to Columbia Ave	McNichol Paving & Cons. Co	10-29-09	5-24-12	122 883.27
109	Street changes, Columbia Ave. to Broad	Am. Paving & Cons. Co	9-21-08	11-23-10	110 756.42
110	Street changes, Allegheny Ave. to Ontario	McNichol Paving & Cons. Co	10-29-09	11-28-11	134 338.60
111	Street changes, Ontario to Hunting Park Ave	McNichol Paving & Cons. Co	10-29-09	11-28-11	43 125.00
115	Street changes, Montgomery Ave	William A. Mundy	11-24-11	2-20-12	2 185.75

In order to provide for the car load freight business of the freight yards during construction, a temporary freight yard was constructed on property of the Railway Company at Nineteenth Street, capable of holding 163 cars, and a temporary engine yard at Wayne Junction. Work on the freight yard was begun in October, 1907, and finished on May 30th, 1908. The total cost of grading, drainage, and paving, exclusive of the track work, was $40 380.40. Work on the engine yard was begun on January 7th, and finished on June 30th, 1908. The total cost of grading, drainage, engine house, coaling station, water tanks, ash pits, etc., exclusive of the track work, was $46 501.60.

On the completion of the elevation of the tracks, the City was credited with the amounts which it had paid for the construction of these yards, less an amount agreed on, representing the depreciation of the tracks, in the case of the freight yard, and of the tracks and buildings, in the case of the engine yard.

Sewers.

In August, 1907, work was begun, practically at the same time, on the construction of by far the majority of the sewer work connected with the change in grades of the streets. The work is described very briefly under the following contracts:

Contract No. 101.—Sewer on Thirteenth Street from Dauphin to Cumberland: This was required to drain the depression caused by the lowering of the intersection at Thirteenth and Cumberland Streets.

Contract No. 102.—Clearfield Street east of Sixteenth; Sixteenth Street from Clearfield Street to Allegheny Avenue, and on Allegheny Avenue to Seventeenth Street: This was the outlet section of the main system for draining the depressed streets in the Tioga District, extending as far north as Nicetown Station.

Contract No. 103.—Allegheny Avenue from Seventeenth Street to Nineteenth Street, and Nineteenth Street from Allegheny Avenue to Westmoreland: This is the second section of the main sewers for draining the Tioga District.

Contract No. 104.—On Nineteenth Street from Westmoreland to Venango, and on Venango from Nineteenth to Twentieth: This is the third section of the main system for draining the depressed streets in the Tioga District.

Contract No. 105.—On Venango Street from Twentieth to Twenty-first, on Twenty-first Street from Venango to Blabon, on Blabon from Twenty-first Street to Hunting Park Avenue; on Hunting Park Avenue from Blabon to the railroad: This is the last section of the main sewers in the Tioga District, and taps the depression on Hunting Park Avenue under the railroad.

The sizes, costs, and other data in reference to the sewers built under Contracts 101 to 105 are shown in Table 3.

In addition to these, sewers were constructed in conjunction with Contracts Nos. 108, 109, 110, and 111, covering the smaller sewers in connection with the work of lowering the streets along the whole line of the work. Table 3 contains a summary of the entire cost of the sewer work, including inlets, connections, etc., and showing for each contract the length of the sewers and the total cost of the sewer work.

The construction of these sewers presented no unusual features. The work was executed in accordance with the standard plans and specifications of the Department of Public Works, Bureau of Surveys, of the City of Philadelphia.

Method of Construction.

The general character of the construction consists: First, of masonry walls and fill, between Green and Brown Streets, between Jefferson and Broad, and between Seventeenth and Indiana and the Richmond Branch, with steel bridges over the streets, carrying four and five tracks, respectively; and, second, a four-track steel viaduct over Ninth Street between Brown and Jefferson Streets.

Work was begun between Columbia Avenue and Broad Street in January, 1907, by diverting the temporary travel to two temporary tracks located as closely as possible to the west side of the 66-ft. right of way. The retaining walls on the east side, with the eastern halves of the bridge abutments, were next built, and a temporary trestle for two tracks was constructed alongside of them; and approximately one-half of the permanent bridges were erected, so that two tracks on the new high level could be placed in operation. Access to this high level was made by the permanent 1% grade from Broad Street and a temporary 2.5% grade from Columbia Avenue to Norris Street. Crib-work was placed along the western edge of the trestle to a height of about 8 ft., and as much fill as possible was made between the crib

and the wall around the trestle before travel was placed on the trestle. On the completion of the temporary tracks on the high level, schedule travel was transferred to them and abandoned on the low level. The western retaining walls and the rest of the abutments were then completed with the fill and the remainder of the bridges. The fill was generally made by carting in from the cross streets before the abutments were finished, and completed after the abutments were built by raising it over them with a clam-shell bucket and derrick. The permanent tracks were then laid on the new fill on the west side, and schedule travel was transferred to them; the temporary tracks on the trestle were removed; the ties, guard-rails, and stringers of the trestle were taken out for use at other points; the fill was completed, and the permanent tracks were laid in their final position. This same method was followed at all other points where masonry and fill construction was used, except that, in the Tioga District, between Seventeenth Street and the Richmond Branch, the method was reversed, the work being started on the west side first.

TABLE 3.—Sizes and Shapes of Sewers, and Costs on Each Contract.

Contract No.	Size, in feet.	Shape.	Invert.	Other particulars.	Cost per linear foot.	Length, in feet.	Total length, in feet.	Total cost.
101	5	Circular	Shale brick.	.....	$24.67	572.80		
	4	"	" "	.....	17.26	512.50	1 085.30	$29 497.98
102	6	"	Stone block	Rock section	27.00	1 070.50	1 070.50	33 861.39
103	6	"	" "	" "	29.00	868.89		
	6	"	Shale brick	" "	30.00	531.11	1 400.00	46 927.39
104	6	"	" "	Reduced cradle.	31.00	88.00		
	6	"	" "	In tunnel	54.00	170.00		
	6	"	" "	Rock section	42.00	297.89		
	5½	"	" "	" "	30.00	550.00		
	5	"	" "	" "	24.00	552.83		
	4¾	"	" "	Earth or rock	20.00	413.67	2 072.39	71 100.92
105	4¾	"	....	" "	21.50	530.40		
	4	"	.....	" "	18.00	610.00		
	3	"	.....	" "	16.50	687.64	1 827.04	36 602.56
108	..	.....	.....	.....	.....	.....	5 155.27	25 913.77*
109	..	.....	.....	.....	.....	.....	2 040.60	14 842.50*
110	..	.....	.....	.....	.....	.....	2 834.50	14 872.97*
111	..	.....	....	.....	.....	.....	820.47	3 059.15*
Totals..	..	.....	.....	.....	.....	.....	18 306.07	$276 678.63

* Does not include cost of inlets or similar details.

In the construction of the steel viaduct between Brown and Jefferson Streets, the foundations for the two outside lines of columns were constructed with the two tracks carrying schedule travel in the

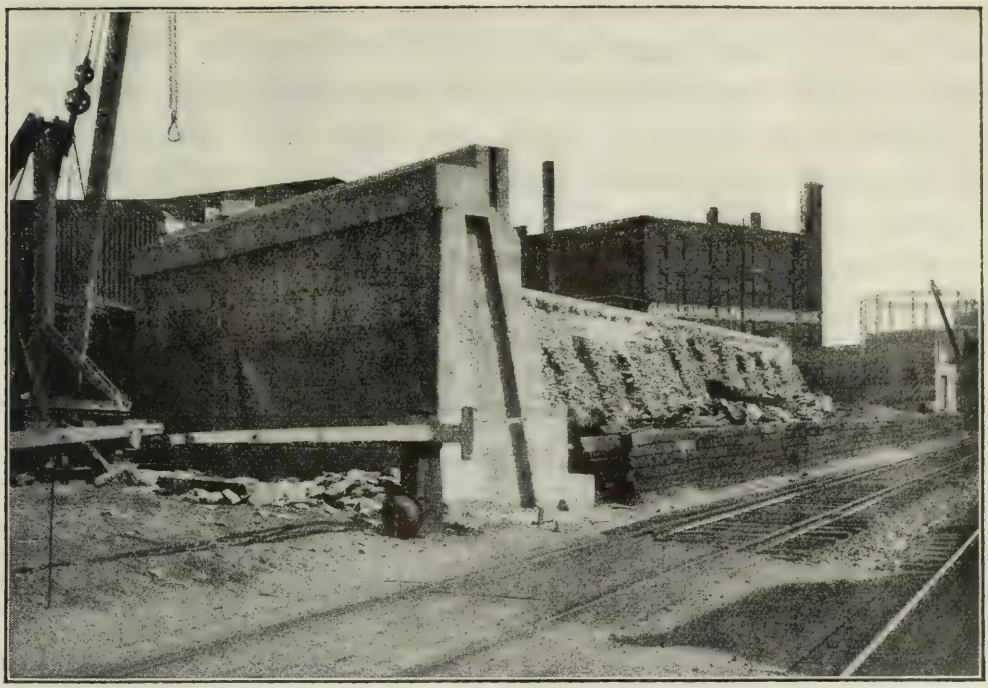


FIG. 2.—ABUTMENT AND DRY STONE PACKING, EAST SIDE.

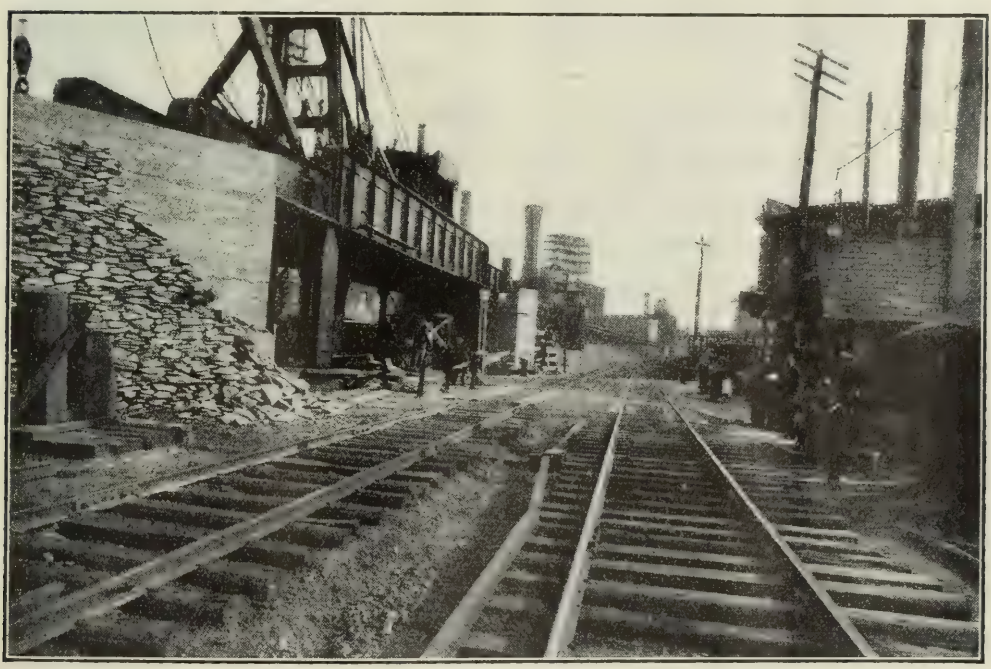


FIG. 3.—ERECTING BRIDGES, EAST SIDE.

center of the 70-ft. street. One track was then placed on the west side of the street, as close to the houses as possible, the other was moved to the east, and the center line of foundations placed. A third track was then built on the west side, and schedule travel was abandoned on the eastern track, which was allowed to remain, to be used in the erection of the eastern half of the steelwork which was next completed. Two permanent tracks were then placed on the completed eastern half of the viaduct, and schedule travel was transferred from the low to the high level. The western side was then erected, completed, and the remaining two tracks were laid.

Temporary Tracks.

Before beginning work, the Railway Company had two tracks on which schedule trains were operated between Green and Cumberland Streets and four tracks from that point to Wayne Junction. There were also two additional tracks used for siding purposes between Girard Avenue and Cumberland Street.

From the time that the main tracks on the street level were disturbed for construction purposes, until the permanent tracks were laid on the high level, all the charges and maintenance were considered as temporary tracks. The instructions of the management were that there must be at all times two tracks in proper condition to operate schedule trains. Only enough material was ordered to carry out the necessary transfers of traffic and to replace that which was not fit for relaying as the tracks were taken up and moved. The general method for providing for temporary traffic has already been described, and it is fair to say that, owing to the care exercised by the contractors, engineers, and the Operating Department, there was no interference with schedule traffic.

Retaining Walls and Abutments.

The part of the construction with masonry walls and fill consisted generally of retaining walls on each side of the 66-ft. right of way, with vertical faces on the lines. It was absolutely necessary to place these walls vertically on the right-of-way lines between Columbia Avenue and Broad Street, as the plans provided for five tracks with 13-ft. centers in this territory, and the same construction was used at other points. As private property adjoined the right of way, there was not room to erect forms for concrete work without trespassing, and large

rubble masonry was indicated and used. Where retaining walls faced on streets, they were built of 1:3:6 concrete with granolithic washed faces and scored with horizontal lines. The same construction was used for bridge abutments. The typical retaining wall was as shown by Fig. 5.

The rubble work (known as third-class masonry) was built under the following specification, the mortar being of 1:3 Portland cement, the stone being bedded in mortar, and the vertical joints grouted with 1:3 grout:

"Third-class masonry shall be formed of approved quarry stone of good shape and good flat beds. No stone shall be used in the face of the walls less than 6 in. thick, or less than 12 in. in their least horizontal dimensions.

"Headers shall generally form at least one-fifth of the faces and backs of the walls, with a similar proportion throughout the mass when they do not interlock, and the face stones shall be well scabbled or otherwise worked, so that they may set close, and chinking with small stones be avoided.

"In walls, 5 ft. thick or less, the stones used shall average from 6 to 8 cu. ft. in volume, and the length of the headers shall be equal to two-thirds of the thickness of the wall; in walls more than 5 ft. in thickness, the stones used shall average 12 cu. ft. in volume, and the headers shall not be less than 4 ft. long. Generally, no stones shall be used having a less volume than 4 cu. ft., except for filling the interstices between the larger stones.

"In no case shall stones be used having a greater height or build than 30 in., and these stones must bond the joints above and below at least 18 in.; in all other cases the smaller stone used must bond the joints above and below at least 10 in.

"The stones in the foundation shall generally not be less than 10 in. in thickness, and contain not less than 10 sq. ft. of surface. The foundation shall consist of 1:3:6 concrete if so directed by the Chief Engineer."

The concrete used was of two kinds, 1:2:4 in bridge seats, pedestal blocks, etc., and 1:3:6 for main walls, foundations, copings, etc. The important and specially interesting points in the concrete specifications were as follows: Stone to be run of crusher, from $\frac{1}{4}$ in. to $1\frac{1}{2}$ in. Sand graded from fine to coarse from $\frac{1}{4}$ in. down. Preference for machine mixing with cubical box type. Consistency to be such that when dumped in place, concrete will not require much tamping, but not wet enough to cause the mortar to

DIAGRAM SHOWING CONSTRUCTION OF MASONRY SECTION

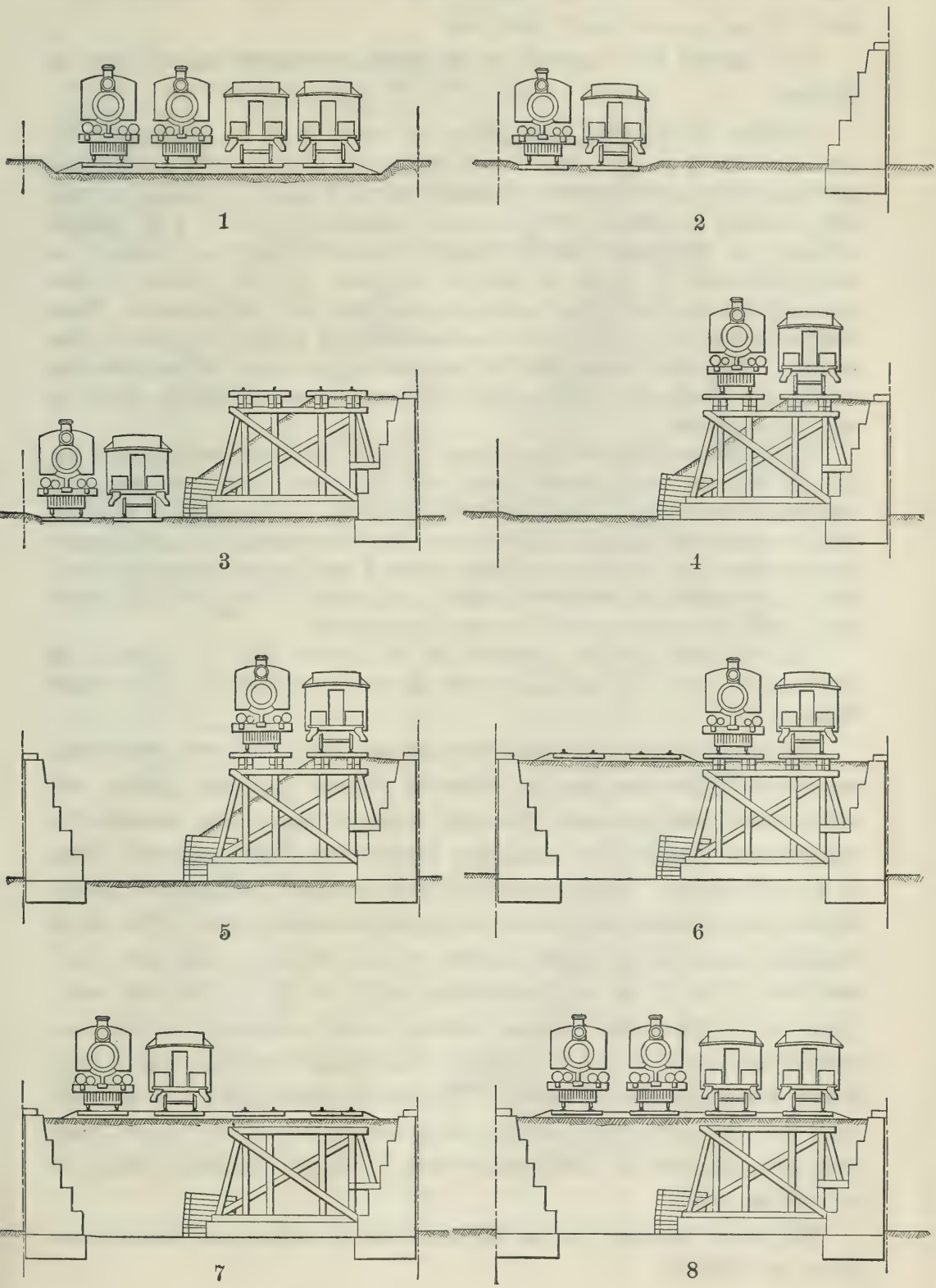


FIG. 4.

separate from the stone. Spaded down and tamped sufficiently to level off and quake freely like jelly.

With special face exposed to the street, the specifications were as follows:

"Surface of concrete exposed to the street shall be composed of 1 part cement, 2 parts coarse sand or gravel, and 2 parts granolithic grit made into a stiff mortar. Granolithic grit shall be granite or trap rock crushed to pass a ¼-in. sieve and screened of dust. For vertical surfaces the mixture shall be deposited against the face forms to a least thickness of 1 in., by skilled workmen, as the placing of the concrete proceeds, and thus form a part of the body of the work. Care shall be taken to prevent the occurrence of air spaces or voids in the surface. The face forms shall be removed as soon as the concrete has sufficiently hardened, and any voids that may appear shall be filled up with the mixture.

"The surface shall then be immediately washed with water until the grit is exposed, and rinsed clean and protected from the sun and kept moist for 3 days. For horizontal surfaces, the granolithic mixture shall be deposited on the concrete to a least thickness of 1.5 in. immediately after the concrete has been tamped and before it has set, and shall be troweled to an even surface, and after it has set sufficiently hard, shall be washed until the grit is exposed.

"All concrete surfaces exposed to the street shall be marked off into courses in such detailed manner as may be directed by the Chief Engineer."

The usual clauses regarding strength and finish of forms were used, and special fillets for the top edges of copings, pedestal blocks, and bridge seats were required. No reinforcement was placed in walls for temperature changes, but expansion joints were placed generally from 30 to 50 ft. apart. These were made with tongued and grooved joints, on the faces of which there were corrugated asbestos boards. The use of embedded stone in the mass concrete was allowed, the stone not to exceed 6 to 8 cu. ft. in the foundations and 2 cu. ft. in the neat work. These were thoroughly cleaned selected stone embedded in the concrete immediately after it was placed. The stones were also used to form dowels bonding each day's work. The use of these stones was optional with the contractor, and though the quantity varied very much, they were generally put in. All copings were of concrete poured in place on top of the walls.

The prices obtained for the various kinds of rubble and concrete are shown in Table 4.

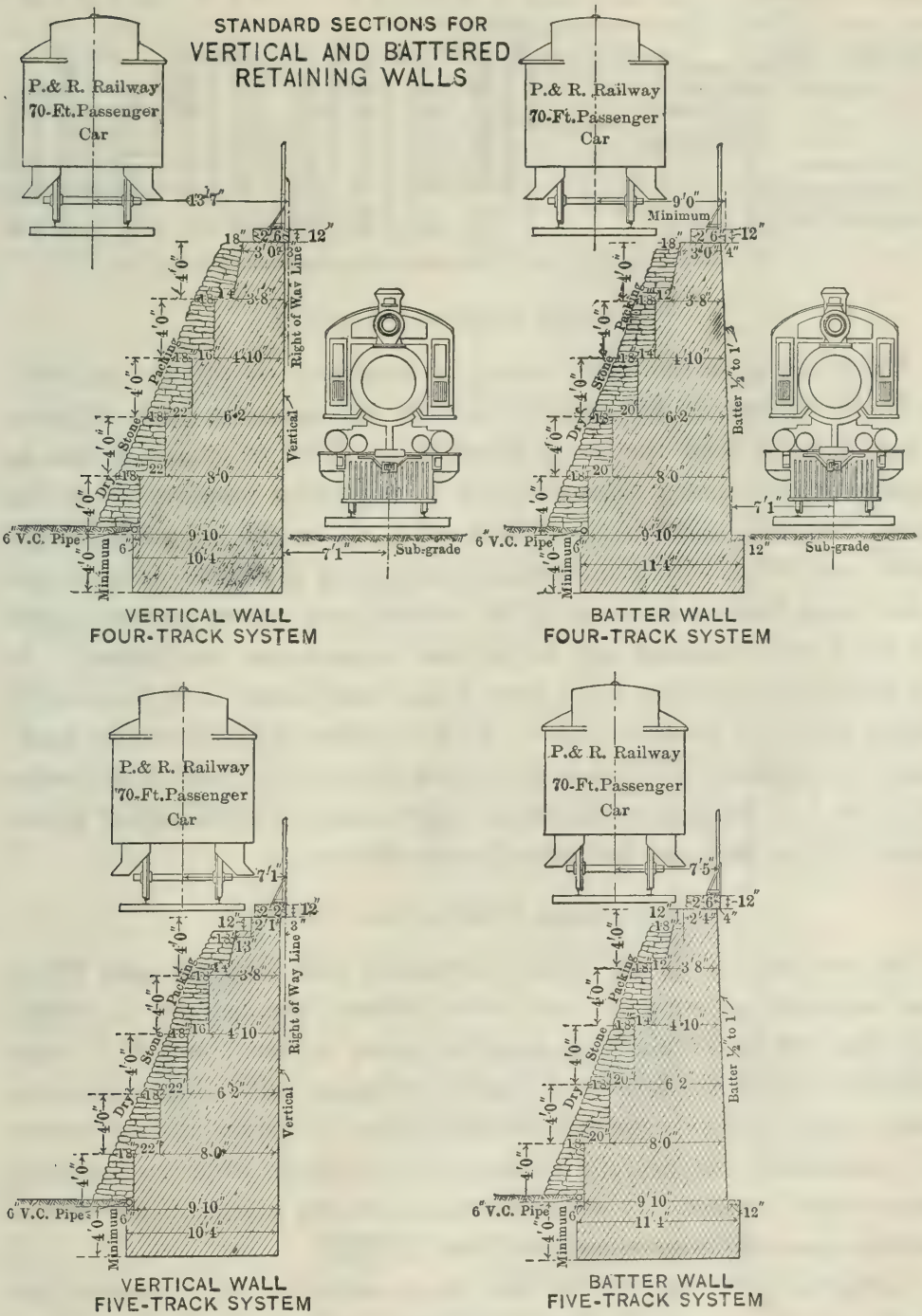

STANDARD SECTIONS FOR
VERTICAL AND BATTERED
RETAINING WALLS
P.& R. Railway
70-Ft.Passenger
Car
Right of Way Line
Vertical
Batter ¼" to 1'
Stone Backing
Dry Rubble
6" V.C. Pipe
Minimum
Sub-grade
VERTICAL WALL
FOUR-TRACK SYSTEM
BATTER WALL
FOUR-TRACK SYSTEM
VERTICAL WALL
FIVE-TRACK SYSTEM
BATTER WALL
FIVE-TRACK SYSTEM

FIG. 5.

TABLE 4.

Contract numbers	1	4	5	7	9	10	12	14	32
Third-class—neat	$6.00	$5.75	$6.50	$5.70	$5.25	$5.70	....	$6.50	....
" " —foundations	6.00	5.00	5.00	5.25	5.50	5.00	....	5.00	....
1 : 3 : 6 concrete—neat work	5.80	6.25	6.45	5.55	6.25	6.25	$6.30	6.45	$6.50
" " —foundations	5.50	5.60	6.00	5.25	5.63	5.50	6.30	6.00	5.75
" " —copings	7.25	7.50	6.75	7.75	7.25	7.00	7.50	6.75	6.75
" " —washed face	6.50	6.50	6.70	7.45	6.40	6.50	6.30	6.70	6.60
1 : 2 : 4 concrete	7.25	7.50	7.75	8.25	7.50	7.50	7.75	7.75	7.75
Dry stone packing	2.50	2.50	3.75	2.90	2.45	2.50	2.75	3.75	2.75
Arch concrete—1 : 2 : 4	....	....	7.50	....	....	....	....	....	....
Unclassified excavation	0.65	0.50	0.50	0.95	0.48	0.50	0.40	0.50	0.90

Drainage Back of Walls.

On account of the walls facing immediately on private property, or on the streets, it was decided to carry the drainage back of them, a step farther than the usual practice, in order to guard as far as possible from leakage through them to adjoining property or to the street, and the entire back of each wall, up to the subgrade of the tracks, was filled with a minimum thickness of dry-stone packing, the stone being from $\frac{3}{4}$ to 2 cu. ft. in volume, and placed by hand after the walls were finished and before the embankment was placed. As the third-class masonry walls were built, their backs were thoroughly dashed with 1 : 3 cement mortar. At the bottom of the dry-stone packing, 6-in. vitrified clay pipes with open joints were laid, with grades of about 1%, leading to the cross streets and there connected to the sewers. This drainage has proved very effective.

Temporary Trestle and Crib.

In the sections of the work constructed with masonry and fill, it was necessary to support two tracks between the bridges on trestle and then fill in as much as possible, using a crib about 8 ft. high with its face 5 ft. from the nearest running rail. The temporary trestle was built on concrete footings so as to reduce the settlement to a minimum and prevent wedging at the top in order to maintain proper track surface. It was constructed for two tracks in accordance with the general design shown on Plate LXVIII.

Long-leaf pine was used in the ties, guard-rails, and stringers, and these were all recovered when the fill was entirely completed and before the permanent track was laid. The caps, sills, posts, and bracing were of short-leaf pine, and remained buried in the fill.

PLATE LXVIII.
TRANS. AM. SOC. CIV. ENGRS.
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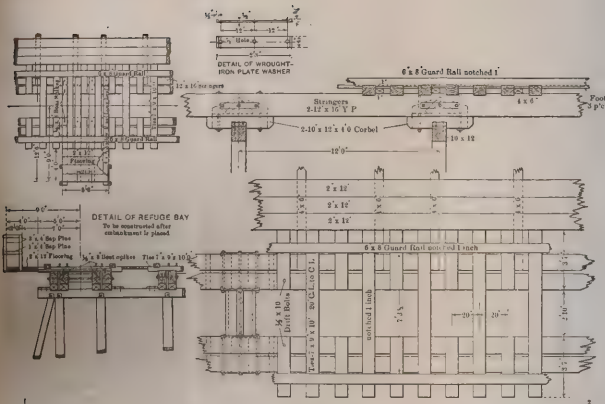
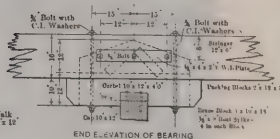
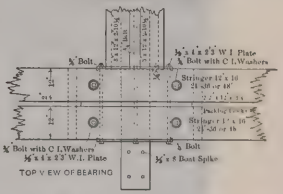
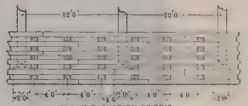
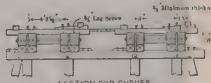
WAGNER ON
TRACK ELEVATION, PHILADELPHIA, PA


DETAIL OF WROUGHT-IRON PLATE WASHER
6 x 8 Guard Rail notched 1"
Stringers 2-12" x 16" I" P
Foot-Walk 3 p'c's-3" x 12"
2-10" x 12" x 4'0 Corbel
10 x 12
DETAIL OF REFUGE BAY
To be constructed after embankment is placed
6 x 8 Guard Rail notched 1 inch
Ties 7 x 9 x 10'0
20'-0"
TYPICAL SECTION OF TRESTLE

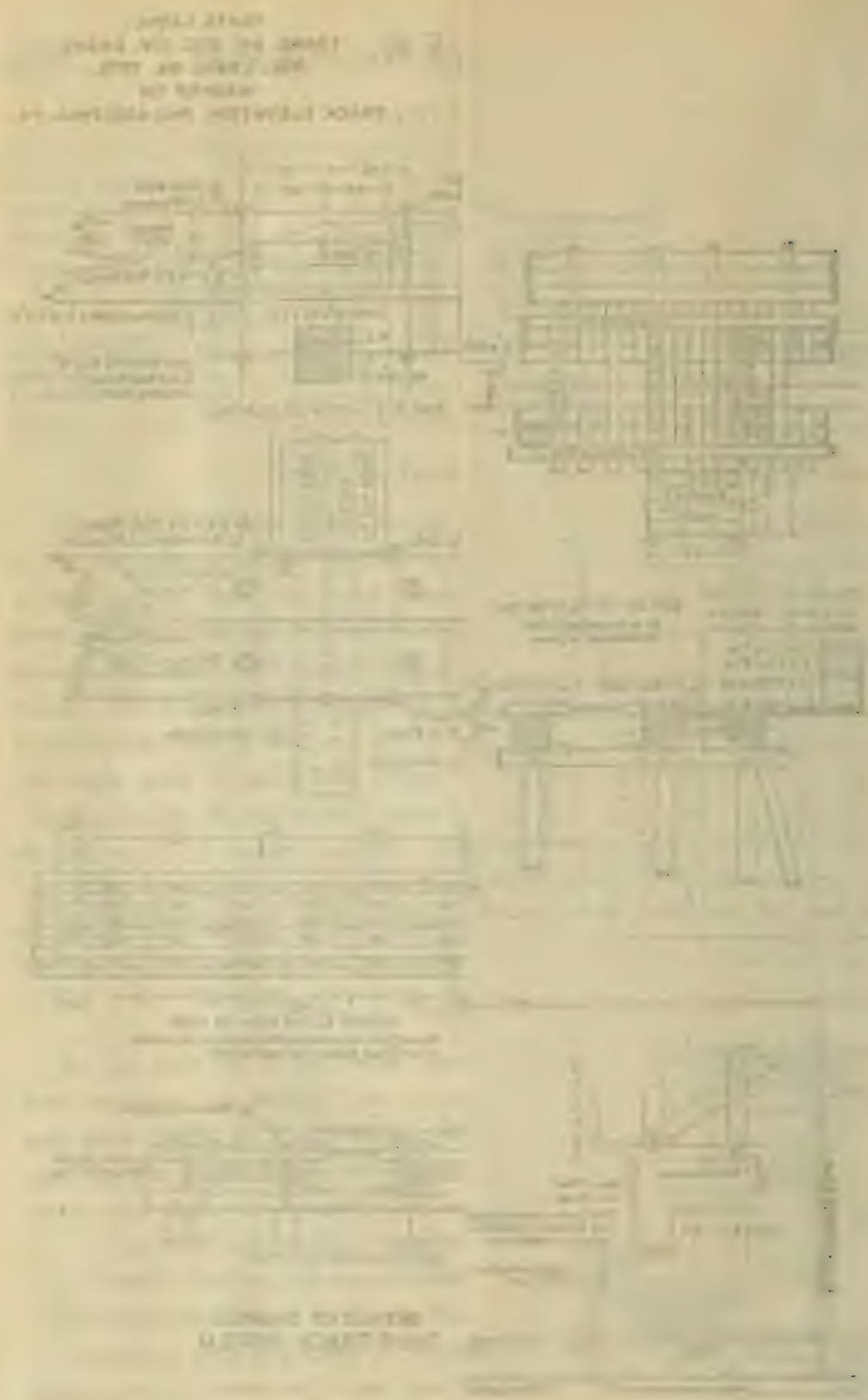

Bolt with C.I. Washers
Stringer 12" x 6"
Corbel 10" x 12" x 4'0"
Pack'ng Blocks 7" x 11" x 3'4"
Cap 10" x 12"
Brace Block 1" x 10" x 15"
END ELEVATION OF BEARING







DETAILS OF TRESTLE
FOUR-TRACK SYSTEM



The prices paid for this work are shown in Table 5, and included everything above the concrete footings ready to lay track.

TABLE 5.

Contract numbers	1	4	5	7	9	10
Long-leaf yellow pine, per 1 000 ft. b. m.	$43.00	$44.50	$40.00	$43.00	$44.50	$42.00
Short-leaf yellow pine, " 1 000 " "	41.00	38.00	36.00	41.00	37.00	38.00
Wrought iron or steel, per pound	0.03	0.03	0.04	0.03	0.03	0.03

The average height of the double-track trestle was about 17 ft., and its average cost per linear foot was about $18.50, exclusive of the concrete footings.

The cribwork was covered by the following specifications:

"Where cribwork is required, it shall cover all temporary lumber work with the necessary filling and tamping required to retain slopes along partly made embankments or at other places, as may be directed.

"The lumber used in cribwork shall be placed as shown on the plans. It shall consist of second-hand lumber which is sufficiently sound to hold the drift-bolts.

"As the lumber is laid and drift-bolted, the embankment shall be filled in and thoroughly rammed. This work shall be paid for at the item price given in the proposal for cribwork per 1 000 ft. board measure, which price shall include all drift-bolts and the placing and ramming of the necessary embankment within the crib."

The prices obtained for the cribwork were as follows:

Contract numbers	1	4	5	7	9	10
Cribwork per 1 000 ft. b. m.	$15.00	$15.00	$20.00	$24.00	$25.00	$20.00

Embankment.

Embankments were built as soon as the footings for the temporary trestle were begun, and all excavated material from the foundations of walls and abutments was placed around them and in the crib, payment being made for excavation only. On the completion of the sections of the trestle between streets, the crib was carried up to its full height, embankment material was delivered on the cross streets, raised over the abutments with derrick and clam-shell buckets, and the fill completed around the trestle to its finished lines.

In some cases where excavation was available from the depressed streets, the trestle was planked, and the street grading contractor hauled the excavated material over it, and dumped it as part of his cost of excavation. With the exception of this, all material going

into embankments was furnished from outside the work and placed by the contractor at fixed prices, measurement being made on carts or wagons on delivery, each wagon load being measured by a representative of the contractor and an inspector who checked at the close of each day's work.

After schedule travel was placed on the temporary trestle, the remainder of the fill was made with teams hauling in from the adjoining streets, and after the abutments were completed the remaining material was raised over the abutment, as was done on the other side.

All material for embankment was inspected carefully and no ashes, cinders, or perishable material were allowed. Nearly all of it came from various building operations throughout the city.

The prices paid for embankment, delivered and placed by the contractor, were as follows:

Contract numbers..	1	4	5	7	9	10	12	14	22
Embankment.....	$0.20	$0.14	$0.45	$0.52	$0.10	$0.15	$0.10	$0.45	$0.175

Bridges.

All the bridges on the improvement carried railroad tracks, there being not a single highway bridge. Wherever possible, they were of deck construction, and, where through girders were required, the top of the girder was kept as low as possible, or the distance between tracks was spread to 14 ft.—the standard being 13 ft. All bridges were of the plate-girder type with solid steel floors either of built-up troughs or buckle-plates. The top of the steel on the floors was protected in all cases by some kind of water-proofing described later. Generally, all spans crossing streets had columns on the curb lines. The underside clearance over the streets established by ordinance was 14 ft. All masonry for supporting the steelwork was executed under separate contracts, the bridge contracts, however, covering all water-proofing and painting ready for the ballast.

The design of all the bridges was complicated seriously by the fact that they had to be planned so as to be erected in two sections, in order to carry travel on the high level on one side while the other side was under construction. This was specially difficult where the skews of the street crossings were great, and, in two cases, it was necessary to erect part of the steel over travel on the low level. The problem of water-proofing was also made more difficult on account of the provision for the double erection and the necessary joint.

FIG. 6.—BUILDING WALLS, WEST SIDE.

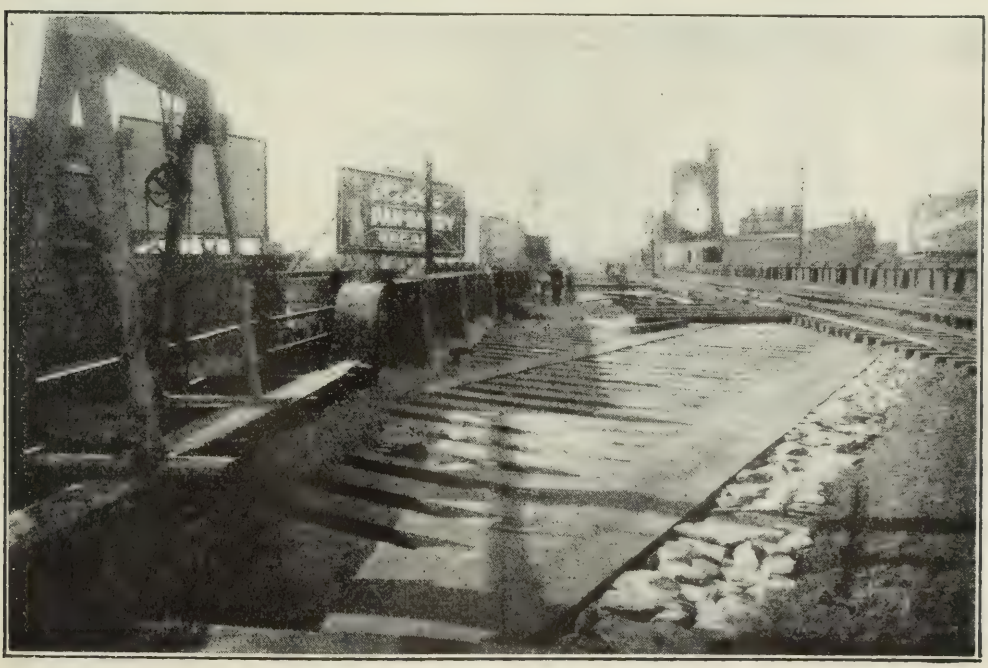


FIG. 7.—FINISHING BRIDGES, WEST SIDE.

1842

TABLE 6.—Data for Bridges.

Location.	Type.	Area, in square feet.	Weight of steel per square foot.	Cost per square foot.	Total cost.
Contract No. 8.—Broad to Norris Streets.					
Norris St. (east)	Deck. 6-in. Z-floor	3 130	115.5	$4.92	$15 400
Norris St. (west)	Deck. 6-in. Z-floor	2 450	130.8	5.60	13 700
Tenth and Diamond	Part deck. Buckle-plate floor... Part half through. 12-in. trough floor	13 325	136.5	5.45	72 600
Susquehanna	Half through. Deep trough floor.	5 160	141.2	5.85	30 200
Colona	Longitudinal trough floor	3 340	126.0	4.74	15 800
Eleventh and Nevada	Half through. 12-in. trough floor.	8 680	157.0	6.40	55 500
Dauphin	Deck. Buckle-plate floor	4 100	117.0	4.61	18 900
Twelfth and York	Half through. Trough floor	7 750	148.5	6.30	48 700
Thirteenth and Cumberland	Half through. Trough floor	11 415	134.1	5.63	64 300
Contract No. 6.—Montgomery to Oxford Streets.					
Oxford St.*	Special half through. Trough floor.	3 740	97.6	$3.50	$13 150
Columbia Ave.*	Special half through. Trough floor.	18 000	103.4	3.55	64 067
Montgomery	Half through. Trough floor	3 700	125.0	4.25	15 740
Contract No. 1.—Jefferson to Green Streets.					
Green St.	Half through. Trough floor	1 660	186.0	$6.20	$10 300
Engine passageway	I-beams and trough floor	3 630	103.0	3.60	13 100
Fairmount (west)	Half through. Trough floor	750	159.0	5.85	4 400
Fairmount (main)	Deck. 6-in. Z-floor	3 000	116.0	3.80	11 400
Fairmount (coal track)	Deck. 6-in. Z-floor	750	110.0	3.87	2 900
Fairmount (east)	Half through. Trough floor	960	134.0	6.10	5 864
Master St.	Deck. Buckle-plate floor	3 050	109.0	4.00	12 200
Contract No. 11.—Seventeenth and Indiana to Richmond Branch.					
Allegheny Ave.	Half through. Trough floor	9 165	153	$4.94	$45 393
Westmoreland	Half through. Trough floor	3 320	125	4.32	14 345
Nineteenth	Half through. Trough floor	7 910	150	4.91	38 920
Ontario	Half through. Trough floor	3 495	120	4.13	14 413
Tioga and Twentieth	Half through. Trough floor	11 070	146	4.81	53 282
Atlantic	Half through. Trough floor	3 535	120	4.10	14 652
Venango	Half through. Trough floor	3 440	149	4.10	14 096
Erie	Half through. Trough floor	7 380	140	4.57	33 697
Hunting Park	Half through. Trough floor	7 195	157	5.08	36 527

*Part of these bridges occupied by passenger station platforms.

At two points only was it possible to construct concrete arches. These will be described separately.

The designs and specifications were of the Philadelphia and Reading standard, which generally provide for a live load about equivalent to Cooper's E-55.

The dead weights assumed for the design of the ballasted floors were as follows, all in pounds per cubic foot: Ballast, 120; concrete, 140; asphalt, 90; lumber, 54.

The impact was by the following formula:

$$I = S \frac{300}{L + 300}$$

I = amount to be added to live-load stress;
S = calculated live-load stress;
L = length, in feet, of the loading which produces maximum stress in the member.

Unit stress = 15 000 lb. per sq. in., in tension.
In compression, 15 000 reduced.

All steel was to be made by the open-hearth process, with a desired ultimate strength of 60 000 lb. for structural steel and 50 000 lb. for rivets; minimum elongation in 8 in. $= \frac{1\,500\,000}{\text{ultimate tensile strength}}$.

Very few of the bridges were distinctly of deck construction, on account of the lack of sufficient head-room, caused by the usual close clearances in grade-crossing work, where the procedure seems to be to squeeze the bridge construction out of existence. Where deck construction was impossible, half through spans were resorted to, with the top flange of the girders as close to the rail level as possible. In a number of cases it was necessary—especially on that part of the structure carrying five tracks—to introduce a deep transverse floor, so as to provide for necessary cross-overs in the tracks. The general type of one of the two styles of construction is shown on Plate LXIX.

Table 6 shows the general data regarding the bridges on four different contracts, and, in examining them, the conditions under which they were designed and the peculiar locations of many of them must be considered.

All columns located on the sidewalks were required by the City authorities to have their base-plates far enough below the surface of the street to show only the main section of the column. All gussets were to be below the surface. The masonry contractors built the floorings up to the level of the base-plates, the bridge contractors, after setting the columns and lining up the steel work, encased the entire base of the columns in concrete which formed part of the surface of the sidewalk and a section of the curb.

Most of the bridges were erected with the usual derrick car, either by jacking it up to the high level, as was necessary in one case, or by

PLATE LXIX.
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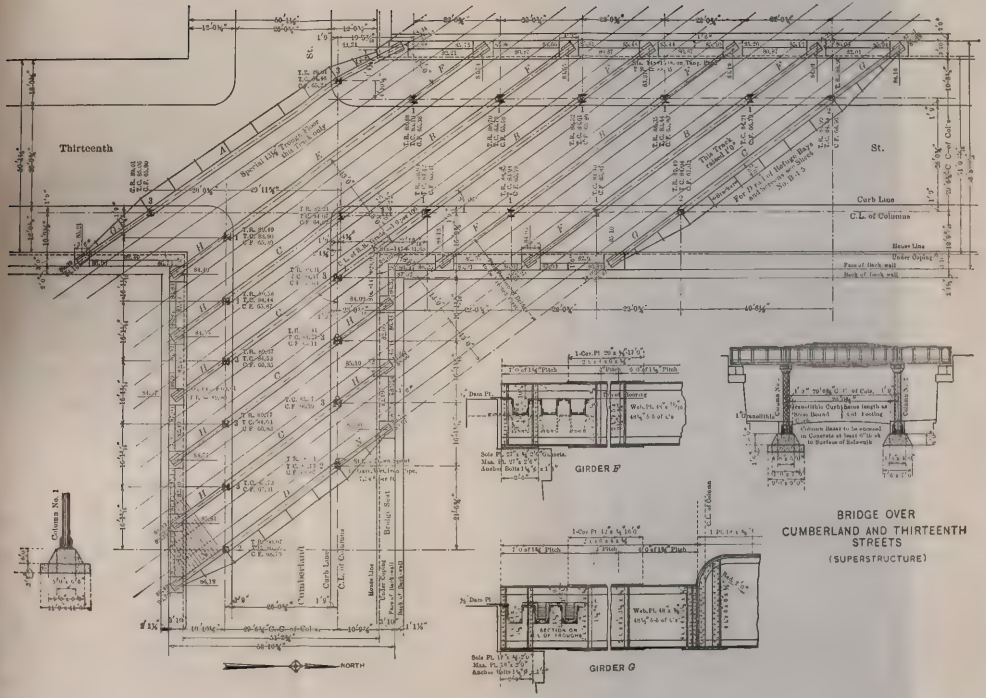
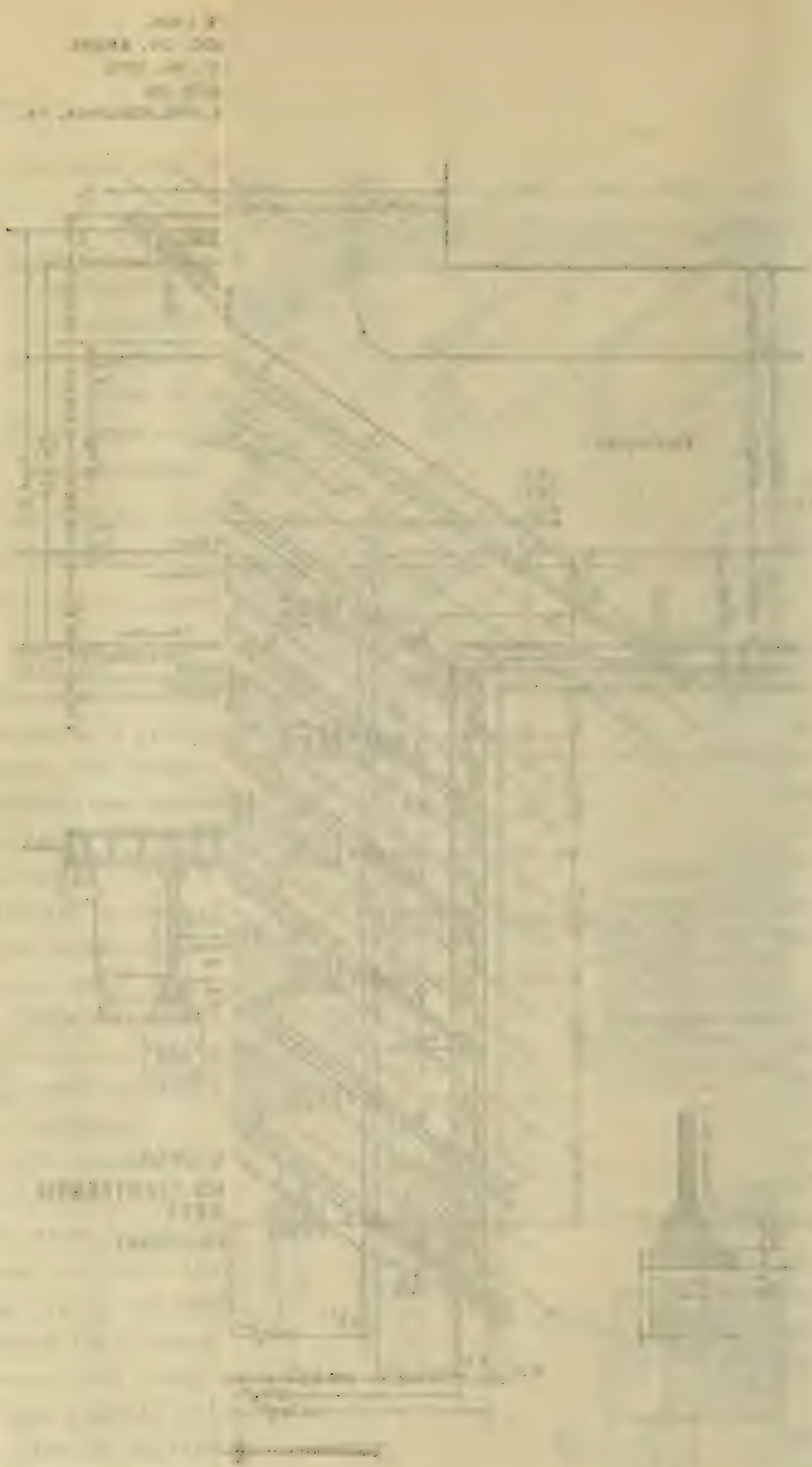
TRACK ELEVATION, PHILADELPHIA, PA.





running it in on the completed high-level approach, in others. When in position to command part of the bridge, the main steel members were run in between schedule trains on the low level, on cars as loaded at the shops, picked off quickly, and either put in place at once, if opportunity offered, or temporarily placed in the street and raised afterward. In some cases the contractors arranged for the railroad wrecker to assist in handling and placing the heavier girders, the time consumed in lifting from the cars and placing in position being very short.

The first halves of the bridges were always located in their permanent positions, no matter what the temporary alignment of the tracks was after leaving them, so that, when once placed, they required no further attention. In erecting the final half of each bridge, the same methods were used.

One of the requirements of the City, in approving the plans of the bridges, was the use of a solid shield or screen over the streets to prevent horses from seeing the moving parts of the locomotives. The height of these shields was finally agreed on as 5 ft. above the top of the rail, and, after considerable study, it was decided to use a cast-iron, ribbed plate, supported on a steel framework attached to the main bridge structure. In a number of cases where there were vacant lots adjoining the streets, these shields were run back on the retaining walls for a distance of 50 ft. or more, depending on the location. In all cases the shields were made part of the bridge contracts, and the costs were included in the lump-sum prices for the bridges.

Viaduct.

Under the terms of the ordinance, Ninth Street was vacated from Brown Street to Fairmount Avenue, and was opened to its full width of 70 ft. from Girard Avenue to Jefferson Street. Prior to the passage of the ordinance, the Philadelphia, Germantown, and Norristown Railroad owned a strip occupied by two tracks between Fairmount Avenue and Girard Avenue, and a right of way, 66 ft. wide, north of Girard Avenue. By the terms of the ordinance, all rights on the street surface owned by this Company were given over to the City, and the Railroad Company retained only the right to enter on the street at all times for renewals or repairs. The ordinance provided for a four-track steel viaduct through this territory, with a paved street

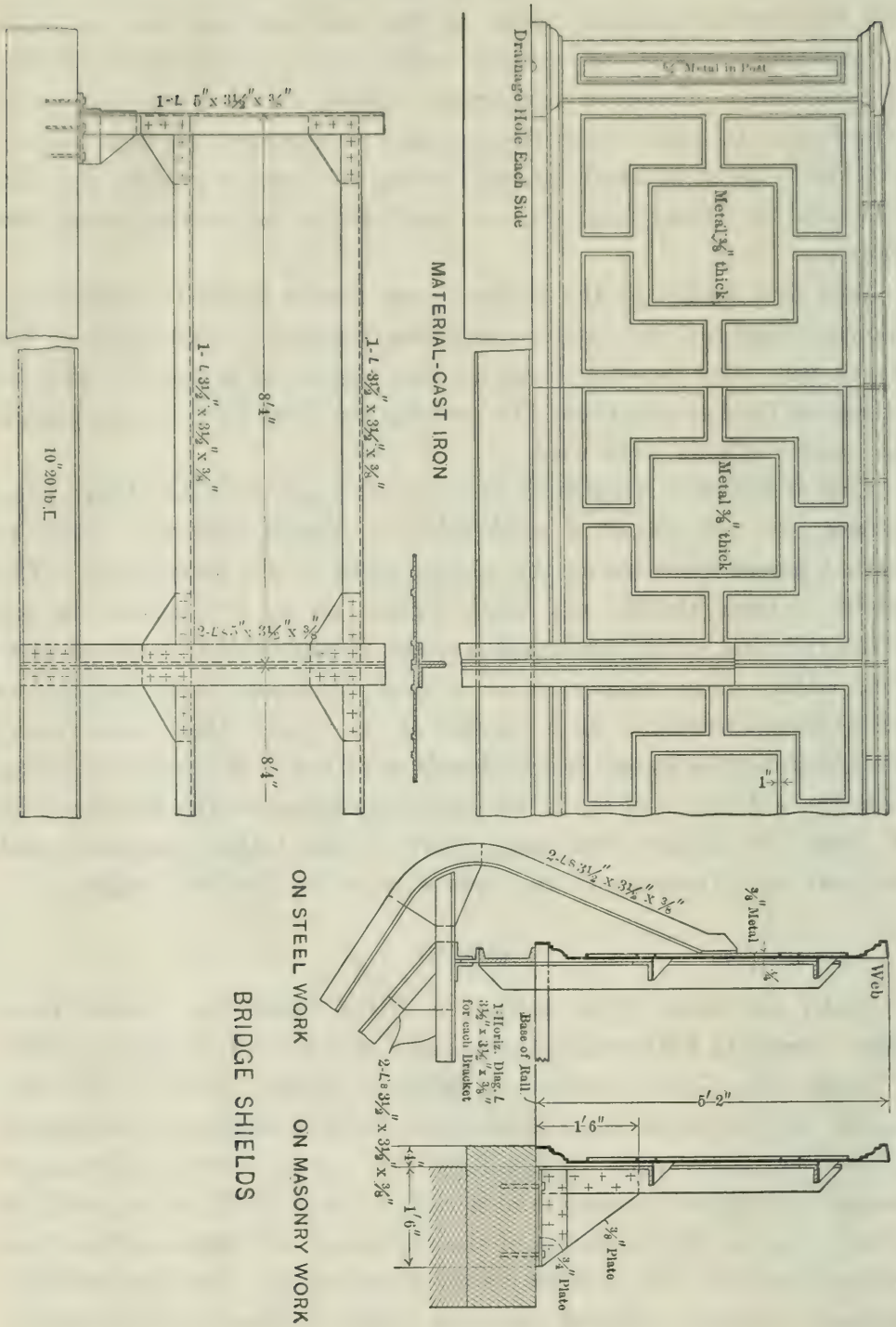

Drainage Hole Each Side
Metal ⅜" thick
Metal ⅜" thick
1"
MATERIAL-CAST IRON
1-L 5" x 3½" x ⅜"
1-L 3½" x 3½" x ⅜"
8'4"
10" 20 lb. I
2-L 3½" x 3½" x ⅜"
8'4"
ON STEEL WORK
ON MASONRY WORK
BRIDGE SHIELDS
2-L's 3½" x 3½" x ⅜"
⅜" Metal
Web
Base of Rail
1-Horiz. Diag. L 3½" x 3½" x ⅜" for each Bracket
5'2"
1'6"
2-L's 3½" x 3½" x ⅜"
1'6"
⅜" Plate
¾" Plate

FIG. 8.

beneath. The Railway Company paid the entire cost of the additional structure for the two extra tracks where it originally had but two.

The structure as built is a four-track steel viaduct supported on three lines of columns, just inside each curb line and in the center of the street. The columns support transverse girders joined over the center column, which in turn carry eight lines of longitudinal

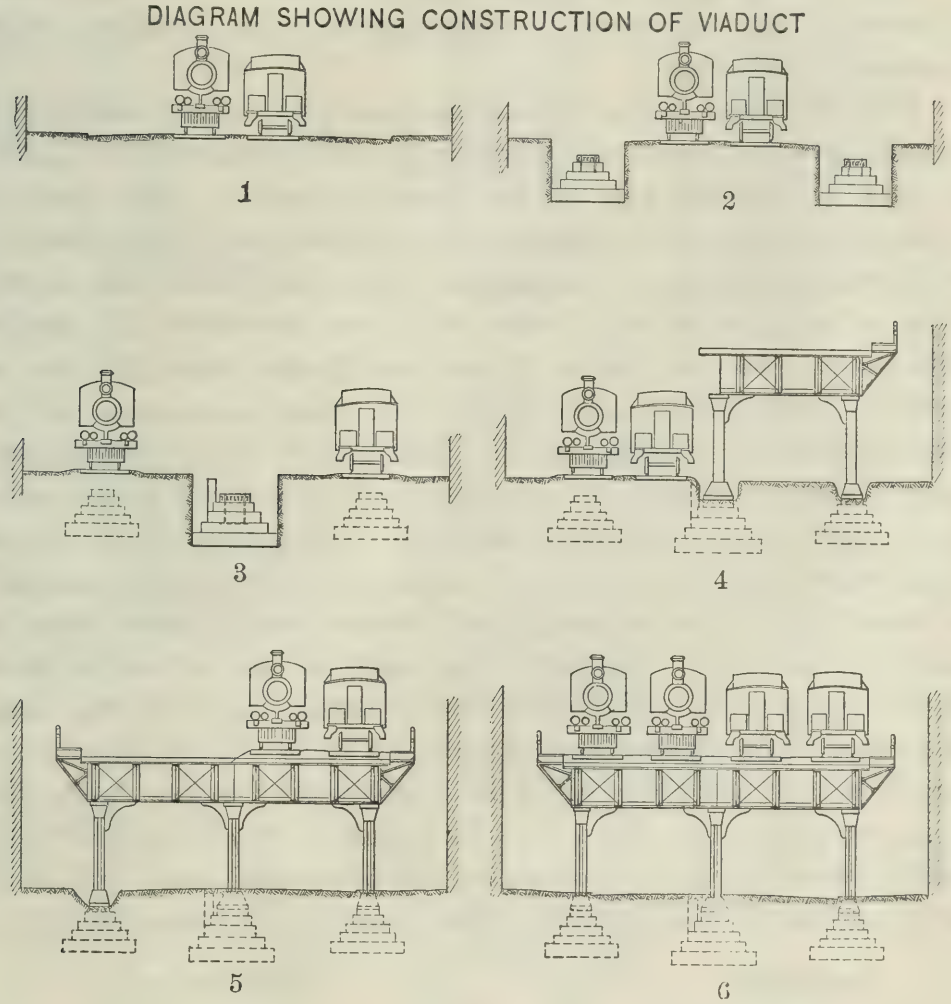


FIG. 9.

girders, spaced 6 ft. 6 in. on centers, and framing into the transverse girders. On the longitudinal girders, and at right angles to them, is a solid floor of 6-in. steel Z-bars and plates, which is cut longitudinally, at about the center of the street, to allow the viaduct to be erected in halves. This floor has its troughs filled with concrete and is covered with 1.5 in. of asphalt mastic in turn supporting the ballast and stand-

ard track. Sidewalks 4 ft. wide are carried on brackets on both sides outside of the dam plates. These have open floors with ½-in. open joints. The structure is designed to allow cross-overs to be placed in the tracks at any point, and the outside line of longitudinal girders is made of the same section as the others, so as to make it readily possible to provide in the future for siding connections.

The foundations for the columns of the viaduct were started under Contract 2, but the contractor abandoning his work, it was relet under Contract 2, Reletting.

With the tracks in the center of the street, the foundations for both outside lines of columns were constructed. The tracks were then moved as close to the west side of the street as possible, which just left room for entrance to the houses on the west side and for the execution of the work on the center foundations. These foundations were troublesome, not only on account of water which was encountered in considerable quantity between Girard Avenue and Brown Street, but also on account of the limited space available for their construction and the close proximity to passing trains. Comparatively little work was done under Contract 2 before it was abandoned, and therefore the prices given are those for Contract 2, Reletting. The bottom of the foundations was stopped between Girard Avenue and Brown Street generally on mixed gravel, and between Girard Avenue and Jefferson Street on gravel or hard clay. Throughout the length of the viaduct the depth of the foundations was controlled to a certain extent by the depth of a sewer which was required in the street before the permanent pavement was laid, and which could not be constructed before the foundations were built. The total cost of the foundation work was $151 631.20, which, reduced to the unit of square feet of viaduct, amounts to $0.76 as the cost of the substructure. The principal unit prices were:

Excavation	$1.65 per cu. yd.
1:3:6 concrete	5.50 " " "

After the completion of the foundations, the eastern half of the viaduct was erected, starting from Brown Street and working north. On account of the very close proximity to passing travel, the erection was a matter requiring the most careful consideration, and the manner adopted was one for which the contractors should receive much credit. On account of the condition of the work at the ends of the

2101
18481.

PLATE LXX
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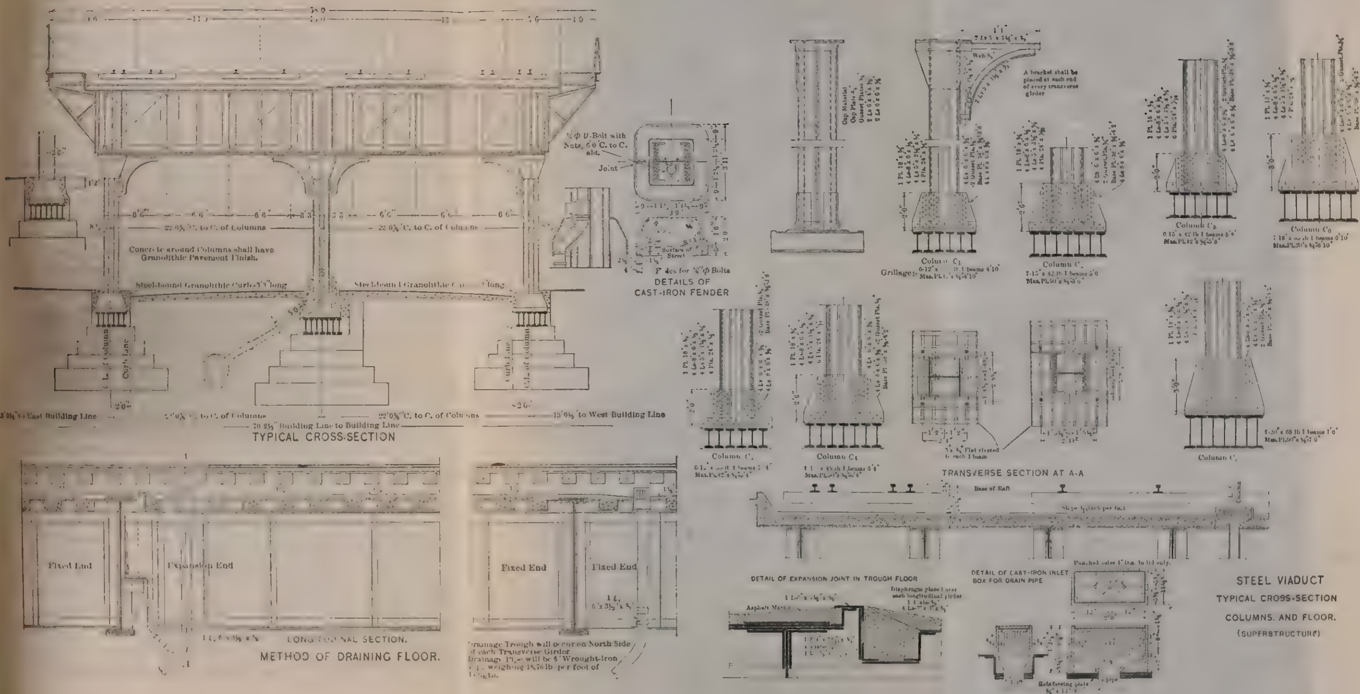
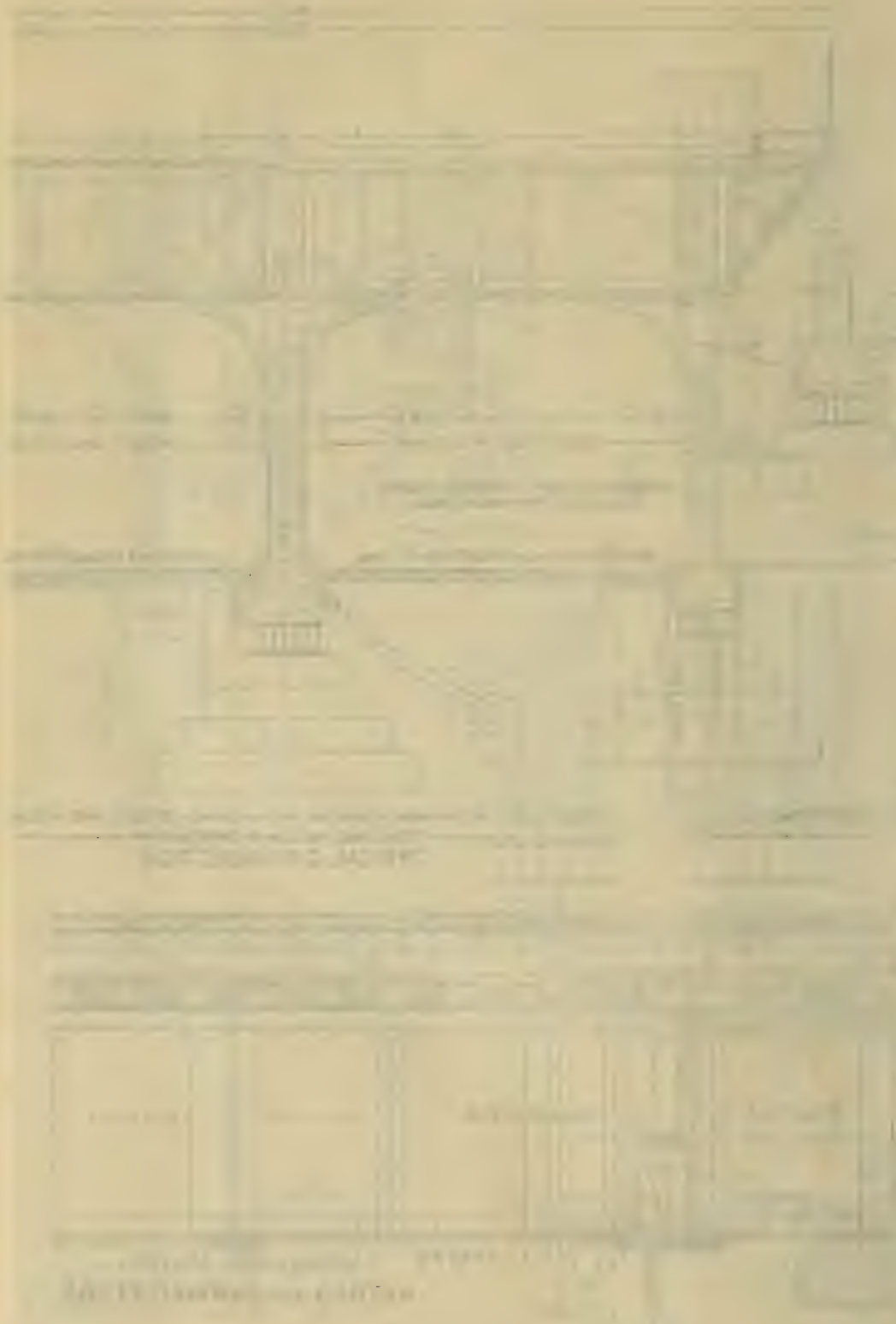


STEEL VIADUCT

TYPICAL CROSS-SECTION

COLUMNS AND FLOOR.

(SUPERSTRUCTURE)



viaduct, a considerable time elapsed between the erection of the first half and the completion of the whole structure. All steel for both sections was brought to the work on the street level, and the rate of progress, which under average conditions was 3 spans per day, was very good. Better time in erection would have been made but for the expansion-joint detail requiring riveting at the transverse girders before other spans could be raised.

In the layout of the viaduct, a standard span of 50 ft. was adopted, and this was used wherever possible, the odd span being placed adjacent to street intersections, which were always special.

On account of the light grades on the viaduct, and the expansion joints at intervals of about 150 ft., it was difficult to arrange for drainage. Low places were provided with cast-iron inlet boxes, to which 6-in. wrought-iron pipes were attached, following down the column and connecting directly with the sewer.

The method of erection of the viaduct planned by the McClintic-Marshall Construction Company was based on:

First. The determination of a method that would come as nearly as possible to being absolutely safe for the passage of trains; and

Second. Economy of time and money.

The method proposed, approved, and adopted was the use of a steel cantilever traveler running on top of the erected viaduct, the erection being accomplished with the aid of two 15-ton traveling electric cranes with one intermediate bridge carrying four 12-ton hoists. The traveler had been remodeled from a steel cantilever traveler previously used for viaduct erection, a few of the truss members having to be reinforced, and an entirely new system of top construction made. The total weight of structural steel in the traveler was 150 tons. The power used was alternating current furnished by the Philadelphia Electric Company, which established connections where convenient, approximately 800 ft. apart. Four heavily insulated flexible wire cables were connected at these stations, and these cables were carried on a reel suspended from the rear of the traveler, the reel being paid out as the traveler advanced, and rewound when new connection was established. The traveler was erected and taken down with a heavy locomotive crane, which also erected enough of the viaduct to provide a platform on which to build the traveler and on which to start work.

This locomotive crane also erected the portion of the viaduct, wider than four tracks, between Brown and Parrish Streets. A 50-ft. boom placed on the front end of the traveler erected the extra width of steel between Master and Jefferson Streets. On the second half, it was necessary to run the traveler on a slight curve between Master and Jefferson Streets, and there was no difficulty in accomplishing this. The traveler had a 70-ft. anchor arm and a 120-ft. cantilever arm. When moving ahead, the cranes were at the extreme rear end of the traveler, thus helping to furnish counterweight against overturning. The traveler was moved with the aid of a heavy hoisting engine placed on a platform on the rear of the traveler, and a single set of double wire-cable falls moved it with ease. The reaction on the wheels at the main point of support, when moving, was very heavy, being 272 000 lb. on the heavier side. After the traveler was moved ahead, and before the crane could move to pick up any load, the columns or front post extension in front of the traveler had to find a bearing on blocking previously placed on the street at the point to which it was being moved. The actual moving, including knocking out the blocking, the moving itself, and the reblocking, did not take more than 15 or 20 min., that is, 15 or 20 min. after picking up iron in one position the traveler could be moved ahead and be ready to pick up iron for a new span; in general, two spans, or about 100 ft., being erected for each move.

The structural steel was shipped from the contractor's Pottstown plant, being loaded on cars in the exact order in which it was wanted, so that, when delivered to the traveler, the first car received contained the longitudinal girders, the next car contained the transverse girders, columns, and cross-frames for the girders, and the third car contained the trough floor, railing, etc., all for the span being erected. These cars were delivered on an abandoned track directly under the work to be erected—the track being approximately midway between the two lines of columns on which the work was being placed. As the material was unloaded the empty cars were dropped down under the viaduct already erected, the overhead clearance permitting this, though in many cases it would not permit a loaded car to pass beneath the viaduct.

The method of erecting in detail was as follows: The rear electric crane would pick up a longitudinal girder and hang it on one of the chain hoists carried on the trolley on the intermediate bridge. The

rear end of the girder would be connected immediately to the transverse girder of the previous bent before the electric crane cut loose—thus the rear electric crane would set the four longitudinal girders. Meantime, the forward electric crane would erect the two columns and pick up the transverse girder preparatory to connecting it to the longitudinal girders being erected. The transverse girder would be brought in against the ends of the longitudinal girders and set on top of columns, there being in general about 2 in. play between the tops of the longitudinal girders and the top flange of the transverse girder, thus not requiring the longitudinal to be hung at the precise height. The four chain hoists supporting the forward ends of the longitudinal girders would then be slacked off, one man to each chain hoist, so that the girders would be seated on the longitudinal girders and the fitting-up bolts entered. The forward crane would then pick up the cross-frames and place them on the longitudinal girders just erected, when both cranes would place the frames in position. The forward crane would then pick up the trough floor, while the rear crane attended to moving ahead any timbers required for the track, these having been snaked ahead with line from the hoisting engine. When handling the 61-ft. girders across Girard Avenue, one crane was used on each end of the girder, this being also necessary on a few of the other special girders. The use of the intermediate bridge, carrying the four chain hoists, obviated the use of temporary bents on which to set the longitudinal girders, and, in addition, was much better, as it readily gave movement in all three directions, as desired, as the bridge was carried on the trucks on the crane runway beams, and each hoist was hung from a trolley running on the bottom flange of an I, and the chain hoists themselves permitted the small vertical movement required for adjustment with little labor.

In general, three spans were raised per day, requiring gangs working ordinarily 6 hours. Where there were four spans, without encountering expansion joints, these were raised in less than 7 hours. In one day it was impossible to raise more steel than between two expansion joints, as the longitudinal girders had to be riveted, as explained previously.

The general figures of interest in connection with the viaduct were as follows:

The contract price was $772 000, including everything from the top

of the concrete footings to the ballast. This included all drainage, water-proofing, and steel grillages for the top of the footings.

Total weight of steel	30 010 000 lb.
Floor area of viaduct	197 300 sq. ft.
Weight of steel per square foot	152 lb.
Cost of water-proofing per square foot	$0.24
Total cost of work per square foot	$3.91

Concrete Arches.

There were only two points on the line where arches over the streets were possible, namely, at Berks and Norris Streets, the streets being 50 ft. wide and the bridges carrying five tracks at Berks and six tracks at Norris Street. These arches were of concrete, and, like all the other bridges, were constructed in two sections longitudinally, and were water-proofed. The general dimensions were as follows:

Berks Street:	Span	50 ft.
	Rise	10 ft. 9⅝ in.
	Thickness at crown	3 ft.
	Width	65 ft. 8 in.
Norris Street:	Span	50 ft.
	Rise	11 ft. 1⅛ in.
	Thickness at crown	3 ft.
	Width	82 ft. 7 in.

At Berks Street, five layers of Genasco Positive Seal Felt in Genasco Compound were used for water-proofing, protected with a layer of 1:3 cement mortar 1 in. thick. At Norris Street, there was used 1.5 in. of asphalt mastic, made by mixing 1 part of Barber Asphalt Company's Compound No. 483 with 4 parts of sand and screenings. This was laid in two ¾-in. layers, each, breaking joints. The compound had the following properties: Bitumen soluble in CS_2, 97%; not more than 0.5% volatile at a temperature of 350° Fahr. for 5 hours; does not flow at 212° Fahr. nor become brittle at — 15° Fahr.; not acted on by H_2SO_4 in 7 hours (25% solution). The water-proofing of both these bridges has been very satisfactory. It cost as follows:

Berks Street	23 cents per sq. ft.
Norris Street	30 " " " "

The prices for the concrete in the arches were as follows:

1:2:4 concrete in arch ring, including centers	$7.50	per cu. yd.
1:3:6 concrete in copings	6.75	" " "
1:3:6 concrete in abutments—street finish	6.70	" " "
1:3:6 concrete in foundations	6.00	" " "

Water-Proofing Bridges and Viaduct.

All the bridges and the viaduct cross City streets or are over stations, as is the case at Columbia Avenue, and it was very desirable that they should be impervious to water, not only for the protection of the public, but also for the preservation of the metal. In a large number of cases, the elevation of the curbs of the streets beneath, and the profile of the tracks above, was fixed within such close limits as to make it difficult to provide proper water-proofing details. The writer believes that one of the most important features of water-proofing design is to get the water away from the structures as fast as possible, and, where the grades of the track are light, this is difficult without using up valuable head-room.

In the first bridges designed and erected, where trough floors were required, the whole of the upper surface of the metal was covered with 1.5 in. of asphalt mastic, applied in two layers to the vertical and horizontal surfaces of the steel floor. This was sloped in the troughs to carry the water to a nipple passing through the steel floor emptying into a wrought-iron trough beneath the floor, and then to down-spouts to the street gutters. If the structure was a deck bridge, the steelwork was first leveled up with Portland cement concrete, made of $\frac{3}{4}$-in. stone and mixed 1:3:6, on which was placed five layers of "Hydrex" or similar water-proofing felt, laid in asphalt compound and protected with a course of hard brick laid flat in the compound. After laying the brick, the compound was poured in the joints and mopped over their tops. The drainage from this type of water-proofing was taken over the back-walls and through the dry packing, already described, to the sewers in the streets. The nipple proved to be a somewhat troublesome detail, and it was difficult to get it tight, the water frequently finding its way around it. These bridges also developed some leaks along the main girders, due to the separation of the mastic from the girder webs; but the leaks were bad in only a few cases, and have been largely corrected by mopping hot pure asphalt on the

mastic along the girder webs, spreading burlap over it, and then applying another mopping of asphalt. The deck bridges with the felt and compound are much tighter.

The second lot of bridges designed and erected differed from the foregoing in that the troughs were filled with cement concrete to the top of the rivet heads, and a smooth coat of asphalt mastic, 1.5 in. thick, was put on in two layers and haunched up against the girder webs and under a flashing angle riveted to the webs. The same method was used on the viaduct. The drainage on the bridges is to one or both ends, and over the back-walls, depending on the grades. On the viaduct, it is to cast-iron inlet boxes and thence through wrought-iron down-spouts to the street level, and in vitrified clay pipes to the sewer. These bridges have generally been tight, except in a few cases where there has been a separation of the mastic from the flashing angles and webs, and these defects have been remedied by cutting out the cracks, filling them with hot pure asphalt, and finally covering them with burlap mopped with asphalt, as previously described. On a portion of the viaduct, the mastic cracked badly on the haunching, and required considerable repairing. This, it is believed, was due to the improper proportioning of the ingredients, as it occurred when the work was first started. In several other cases, leaks have occurred where it was necessary during construction to push the work irrespective of the weather conditions, and in other places where it was difficult to make a good joint between old and new work. It was found to be very difficult to clean properly the surfaces on which either the new compound or the new mastic was added, and it is thoroughly believed that without adequate cleaning and drying it is impossible to bond surfaces with any asphaltic water-proofing material. The necessity of using an asphalt material which has a very low brittle point and a sufficiently high melting point simply to prevent running in hot weather, has been apparent in this work, and is indicated especially for similar work where the structure is subject to vibration. Recent work has also shown the necessity of using a joint, between the asphalt mastic and the metalwork of the structure, consisting of a pocket of pure elastic asphalt.

The estimated costs of the water-proofing were obtained by a careful analysis of the lump-sum figures for the bridges, and show a

FIG. 10.—WATER-PROOFING BRIDGES: FORMS FOR MASTIC.

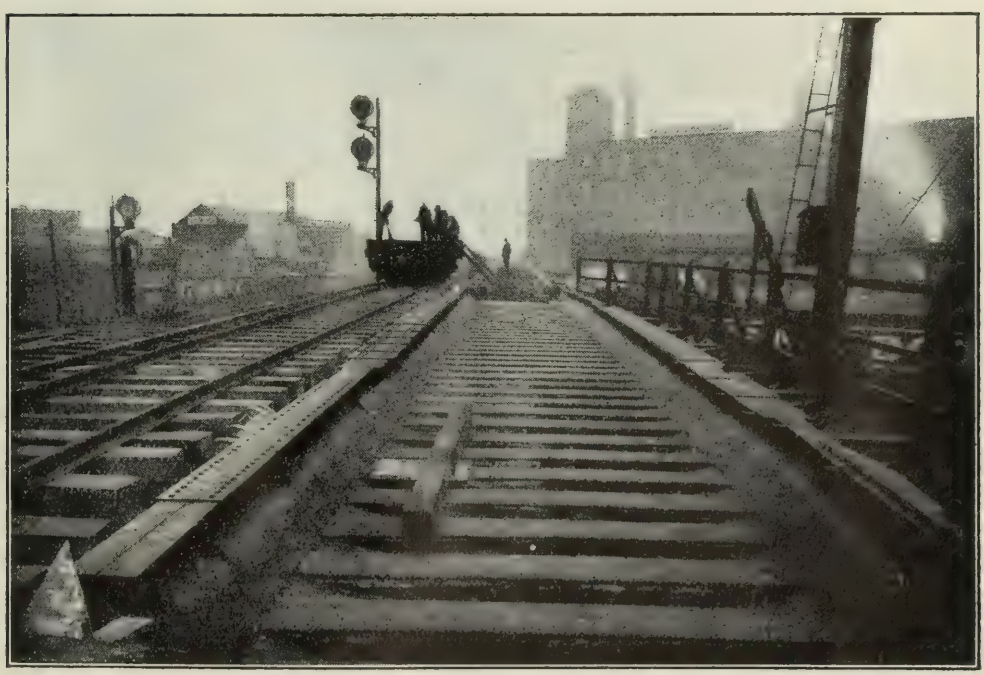
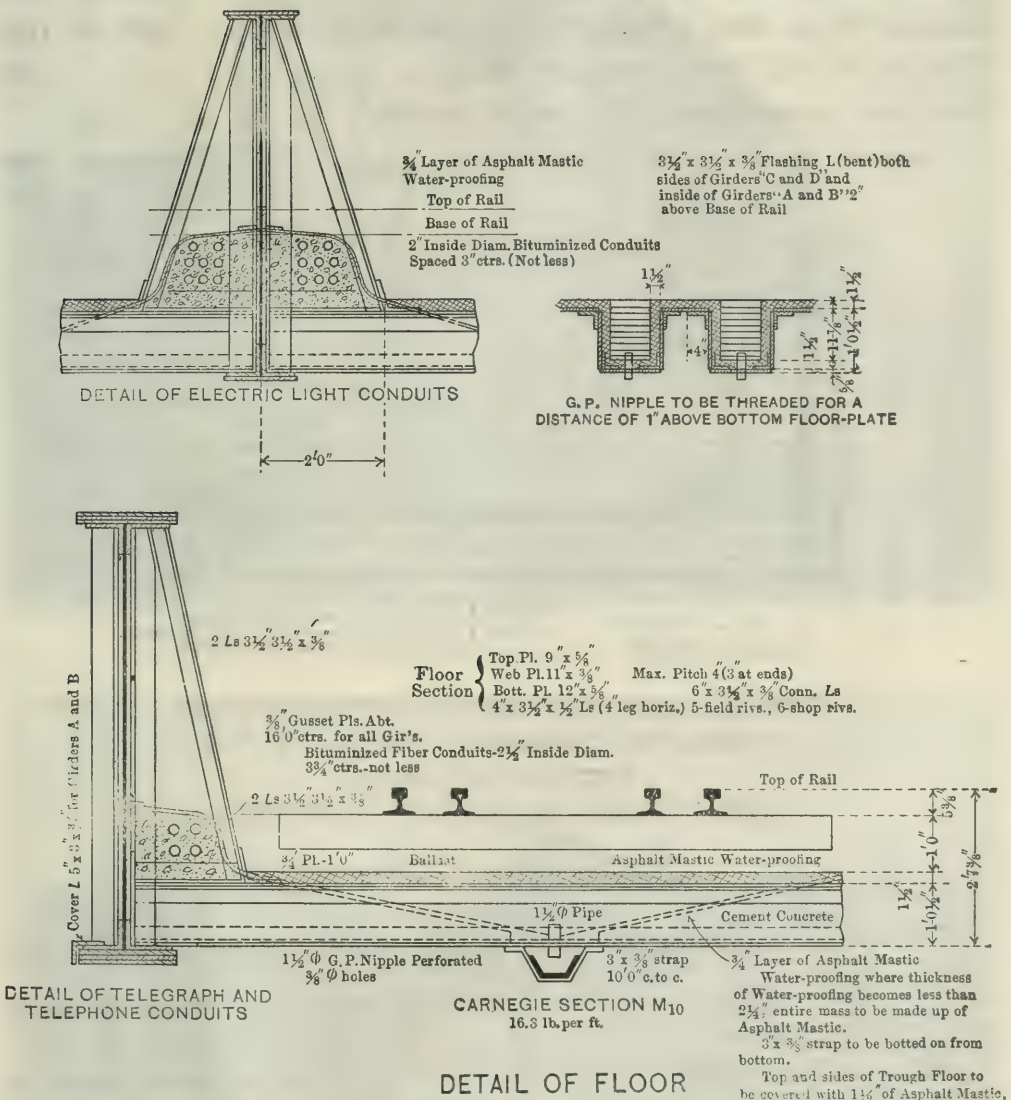


FIG. 11.—WATER-PROOFING BRIDGES, SUSQUEHANNA AVENUE.

1956

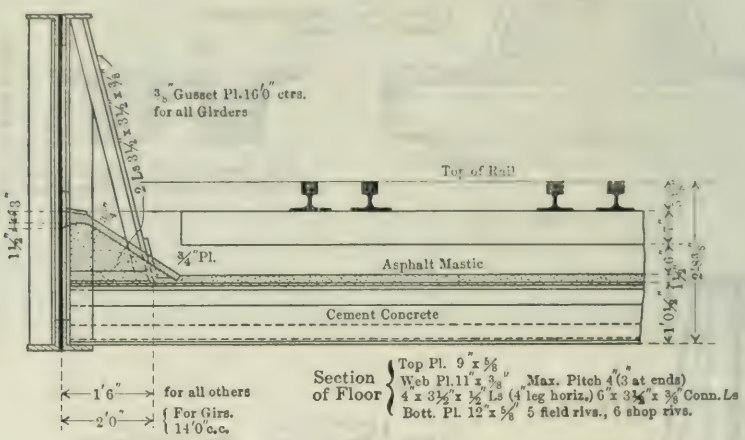

¾" Layer of Asphalt Mastic Water-proofing
Top of Rail
Base of Rail
2" Inside Diam. Bituminized Conduits Spaced 3" ctrs. (Not less)
3½" x 3½" x ⅜" Flashing L (bent) both sides of Girders "C and D" and inside of Girders "A and B" 2" above Base of Rail
DETAIL OF ELECTRIC LIGHT CONDUITS
2'0"
1½"
4"
1½"
G. P. NIPPLE TO BE THREADED FOR A DISTANCE OF 1" ABOVE BOTTOM FLOOR-PLATE
2 Ls 3½" 3½" x ⅜"
Floor Section
Top Pl. 9" x ⅝"
Web Pl. 11" x ⅜"
Bott. Pl. 12" x ⅝"
4" x 3½" x ½" Ls (4" leg horiz.,) 5-field rivs., 6-shop rivs.
Max. Pitch 4" (3" at ends)
6" x 3½" x ⅜" Conn. Ls
⅜" Gusset Pls. Abt. 16'0" ctrs. for all Gir's.
Bituminized Fiber Conduits-2½" Inside Diam. 3¾" ctrs.-not less
Top of Rail
¾ Pl.-1'0"
Ballast
Asphalt Mastic Water-proofing
1½" φ Pipe
Cement Concrete
1½" φ G. P. Nipple Perforated ⅜" φ holes
3" x ⅜" strap 10'0" c. to c.
¾" Layer of Asphalt Mastic Water-proofing where thickness of Water-proofing becomes less than 2¼" entire mass to be made up of Asphalt Mastic.
3" x ⅜" strap to be botted on from bottom.
Top and sides of Trough Floor to be covered with 1½" of Asphalt Mastic.
DETAIL OF TELEGRAPH AND TELEPHONE CONDUITS
CARNEGIE SECTION M10
16.3 lb. per ft.
DETAIL OF FLOOR

TYPICAL CROSS-SECTIONS OF WATER-PROOFING WHERE IT WAS IMPOSSIBLE TO TAKE CARE OF DRAINAGE OFF THE ENDS OF THE BRIDGE.

FIG. 12.

cost per square foot for all water-proofing work and drainage as follows:

	Cost per square foot.
Special deep floors with nipples; 1.5 in. of asphalt.....	$0.42 to $0.55
Hydrex felt on deck bridges; five layers and brick.....	0.25 to 0.26
Viaduct; 6-in. Z floor; 1.5 in. of mastic.....	0.24
Standard trough floor, filled with concrete; 1.5 in. of asphalt	0.24 to 0.30


⅜" Gusset Pl. 16'0" ctrs. for all Girders
Top of Rail
¾" Pl.
Asphalt Mastic
Cement Concrete
1'6" for all others
2'0" For Girs. 14'0" c.c.
Section of Floor
Top Pl. 9" x ⅝
Web Pl. 11" x ⅜" Max. Pitch 4" (3" at ends)
4" x 3½" x ½" Ls (4" leg horiz.) 6" x 3½" x ⅜" Conn. Ls
Bott. Pl. 12" x ⅝" 5 field rivs., 6 shop rivs.

TYPICAL CROSS-SECTION OF WATER-PROOFING WHERE THE DRAINAGE IS TAKEN OFF THE ENDS OF THE BRIDGE. THE TROUGHS FILLED WITH CONCRETE.

FIG. 13.

The specifications for this work may be briefly abstracted as follows:

"Wherever called for on the plans, the decks of the bridges shall be protected as follows:

"First.—By placing 1:3:6 concrete, ¾-in. stone or gravel, troweled on top, as shown on the plans. The cement concrete thus placed shall be allowed to dry thoroughly so as to prevent the formation of steam when the asphalt mastic is applied. All vertical surfaces of concrete or steel shall be prepared by first painting with the asphalt hereafter described, melted and diluted with 62° Baumé naphtha to the proper consistency. The paint shall not be applied to horizontal surfaces except where specially directed.

"Second.—The cement concrete thus prepared shall be water-proofed with asphalt mastic equal in quality, for the intended pur-

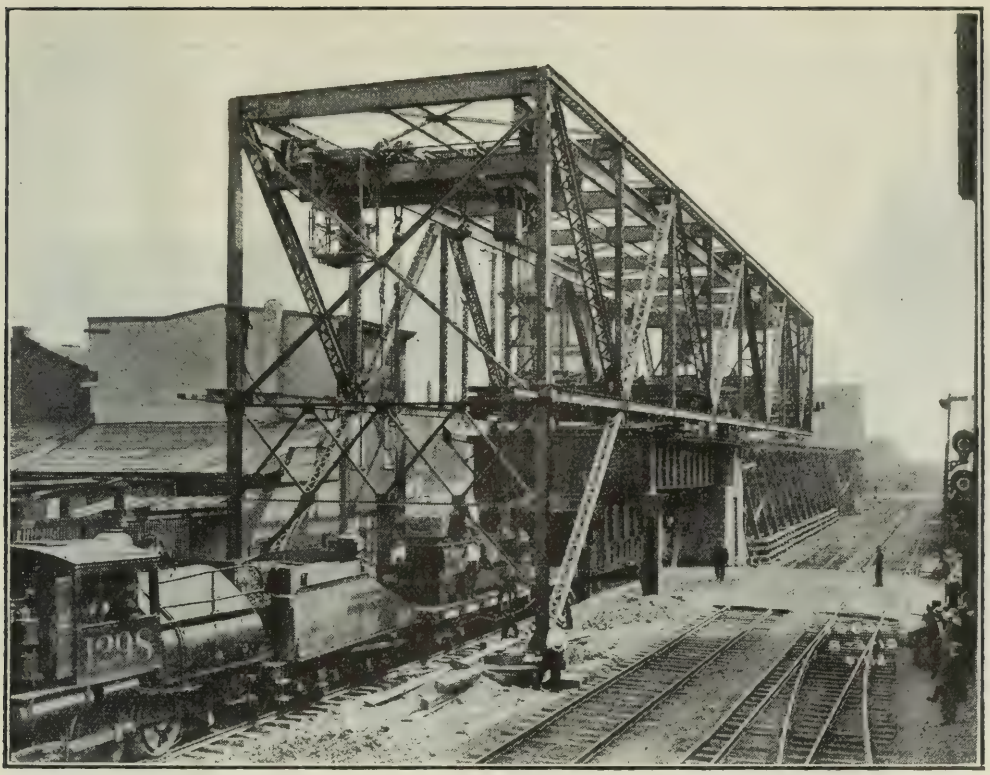


FIG. 14.—TRAVELER STARTING ERECTION OF VIADUCT.

FIG. 15.—EAST HALF OF VIADUCT COMPLETED.

1861

FIG. 16.—ERECTING WEST HALF OF VIADUCT.

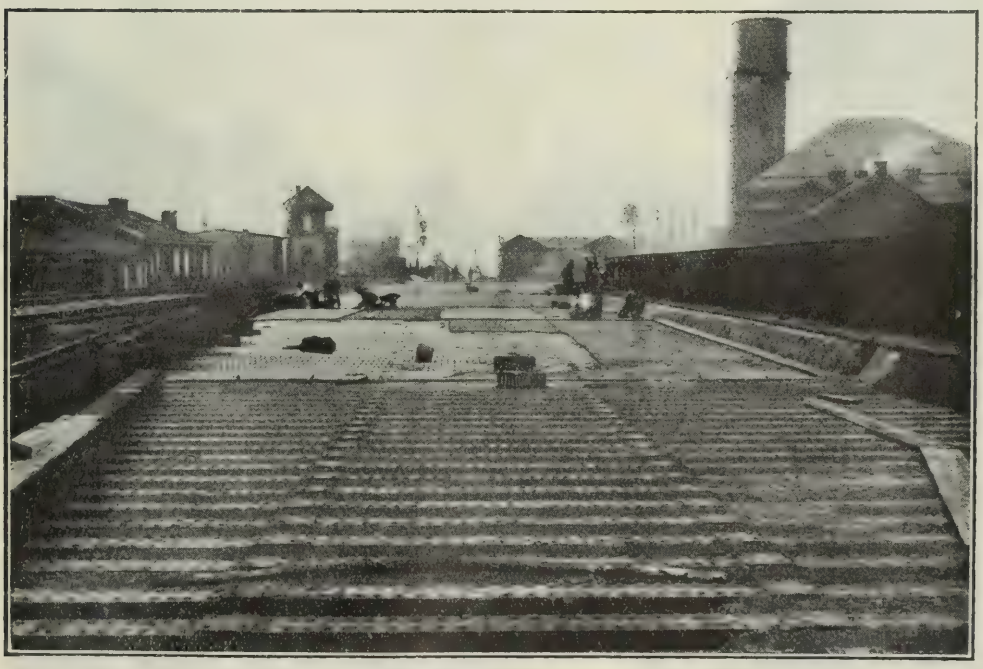


FIG. 17.—WATER-PROOFING SOUTH END OF VIADUCT.

1863

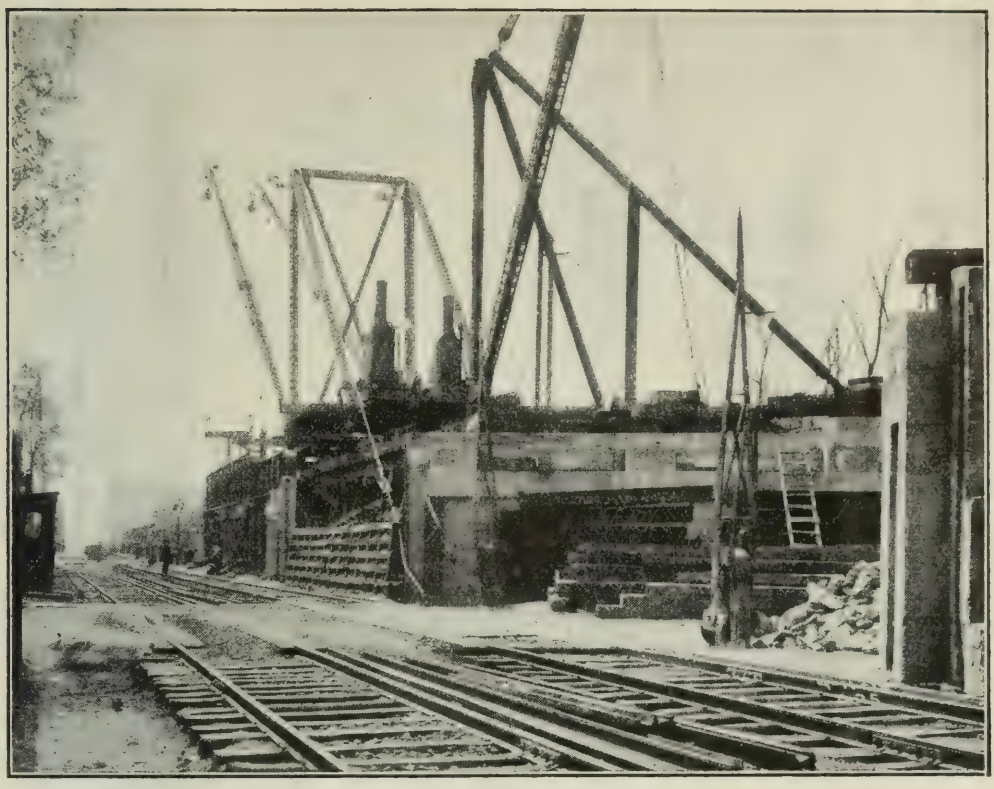


FIG. 18.—ERECTING BRIDGES, TIOGA SECTION, WEST SIDE.

FIG. 19.—WATER-PROOFING BRIDGES, TIOGA SECTION.

1869

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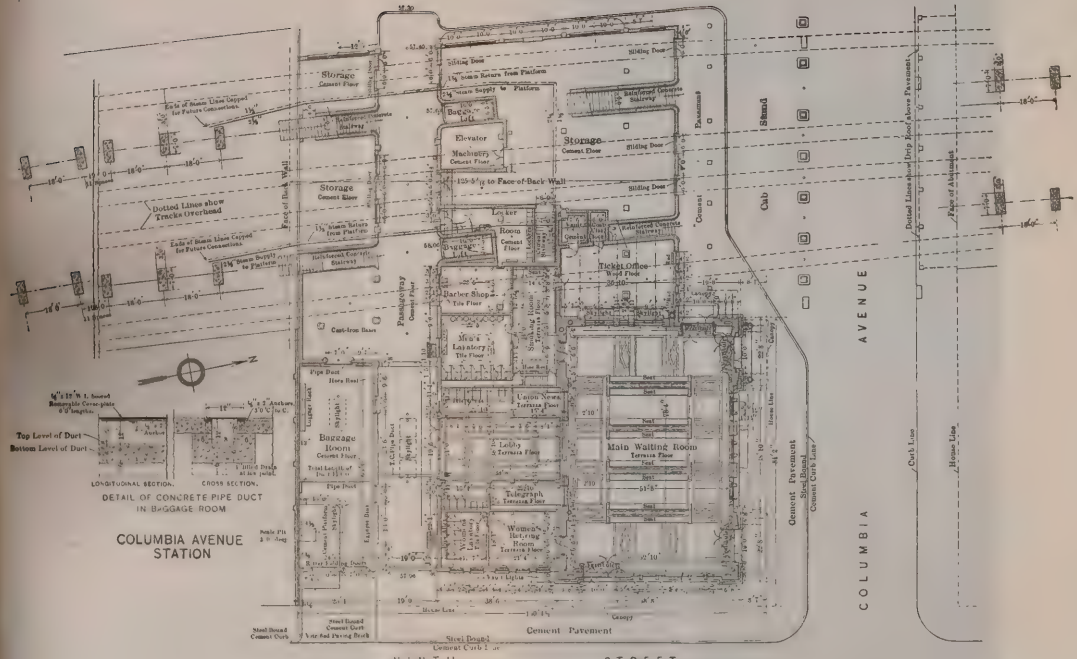
1864A



pose, as to ingredients used and resistence to water, to the following specifications, and be approved as such:

"Neufchatel, Seyssel, or Sicilian rock mastic... 60 parts.
"Clean sharp grit and sand to pass a sieve of
8 meshes per inch..... 30 "
"Refined Bermudez or Trinidad asphalt..... 10 "

"These proportions shall be varied where required by special conditions on the work.

"The mixture shall be made at the site of the work, and heated to a temperature of from 250 to 300° Fahr., and stirred until all the ingredients are thoroughly incorporated. It shall then be spread and thoroughly worked, to free it from voids, and ironed to a smooth surface with hot smoothing irons, if so directed. All mastic shall be applied to the work in two coats, making the total thickness shown on the plans. The coats shall break joints, and the mastic shall be distributed evenly. Where the thickness of the water-proofing materials is less than 2.25 in., the full thickness shall be made up of asphalt mastic.

"Great care shall be taken around expansion joints, drain pipes, and other similar places, and the methods to be followed shall be as specified on the plans or as directed by the Chief Engineer.

"The cost of the water-proofing specified herein shall be included in the lump-sum prices for the bridges."

The asphalt mastic used on this work was prepared from Sicilian Natural Rock Mastic, which averaged from 13 to 16% of bitumen soluble in carbon bisulphide.

Passenger Stations.

Four passenger stations required rebuilding on account of the elevation of the tracks, namely, Girard Avenue Central, Columbia Avenue, Tioga, and Nicetown. As all have been built considerably larger than the old stations, the greater part of the cost has been borne by the Railway Company, the City simply paying one-half of what it would have cost to raise to the new track level structures with the accommodations of those existing at the time the work was started.

Girard Avenue Central.—The four-track viaduct, with its straight alignment at this point, made it possible to provide platforms only on the two outside or local tracks, the station buildings on each side being erected on a steel structure over the bed of Girard Avenue supported by columns on the curb lines. The lengths of the platforms at this point are controlled by existing industrial sidings, and, un-

fortunately, are not as long as desired. This station had very deep foundations for the columns, they being in the bed of an old creek, and cost $54 602.40, the entire cost being assumed by the Railway Company, as the old building was very small.

Columbia Avenue.—This station is an important one, as all trains on the main line and to New York stop there. The tracks cross the street at about the elevation of the old street, making a very severe change of grade. A number of studies were made for the new station, involving very different schemes, all of which, however, included the spreading of the tracks so as to place island platforms between the north- and south-bound tracks, and in so doing entailed the purchase of additional right of way. It was finally decided to purchase a lot at the southwest corner of Ninth and Columbia Avenues, and place on it the main waiting- and baggage-rooms, locating the other necessary facilities beneath the tracks, which are carried on a steel structure for this purpose.

The main building has a frontage of 170 ft. on the south side of Columbia Avenue, 150 ft. on the west side of Ninth Street, and 150 ft. on the east side of Hutchinson Street. The exterior walls are of gray brick with granite base, and Indiana limestone and terra cotta trimmings. The passageway to the trains and the stairways to the elevated platforms are lined with white enameled brick. The interior is designed with complete accommodations for handling heavy traffic. The main waiting-room is 52 ft. by 79 ft., and is 35 ft. high, with polished Italian marble wainscot 9 ft. high. The floors are of marble terraza with marble panels. A ticket office, smoking-room, men's toilet, barber shop, women's waiting-room, telegraph and telephone service, news stands, baggage- and express-rooms, are also provided.

The two platforms on the high level are 800 ft. long. They are of concrete with concrete curbs on the earth-filled portion and with concrete curbs with asphalt mastic over the steel portion. They are covered with heavy sheds of the umbrella type. The entrance stairways to the platforms are from a passageway at right angles to and beneath the track, and between the main waiting-room and the baggage-room. This passageway can be reached from the main waiting-room or from either Ninth or Hutchinson Streets directly. There are exit stairways from each platform, with turnstiles. These lead to Columbia Avenue and also to Oxford Street.

1867

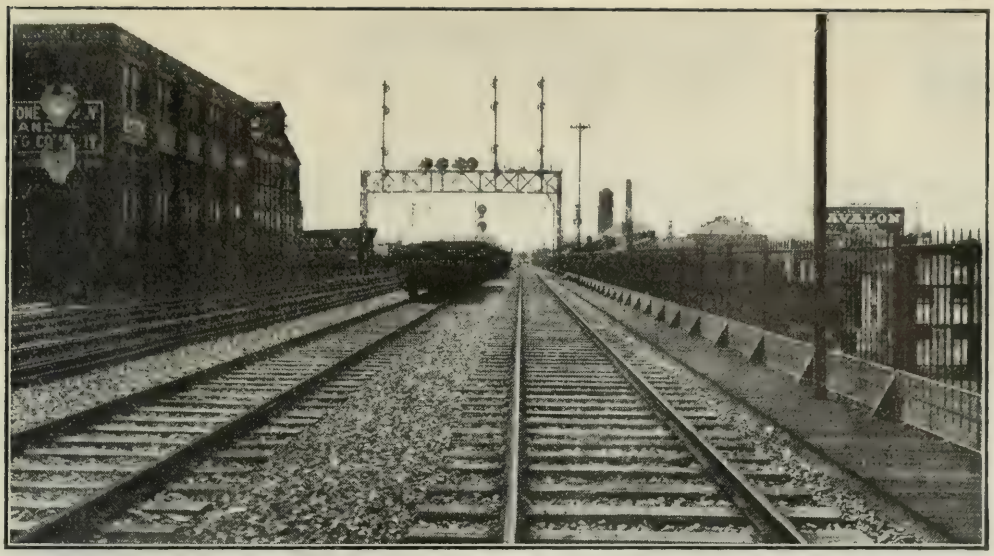


FIG. 20.—COMPLETED VIADUCT FROM ABOVE.

FIG. 21.—OLD STONE TIES, P., G. & N. R. R., IN PLACE.

FIG. 22.—ENGINE "ROCKET" IN COLUMBIA AVENUE STATION.

1869

FIG. 23.—COLUMBIA AVENUE STATION.

FIG. 24.—INTERIOR VIEW, COLUMBIA AVENUE STATION.

1871

FIG. 25.—GIRARD AVENUE STATION.

FIG. 26.—TIOGA STATION.

1872

1274 A

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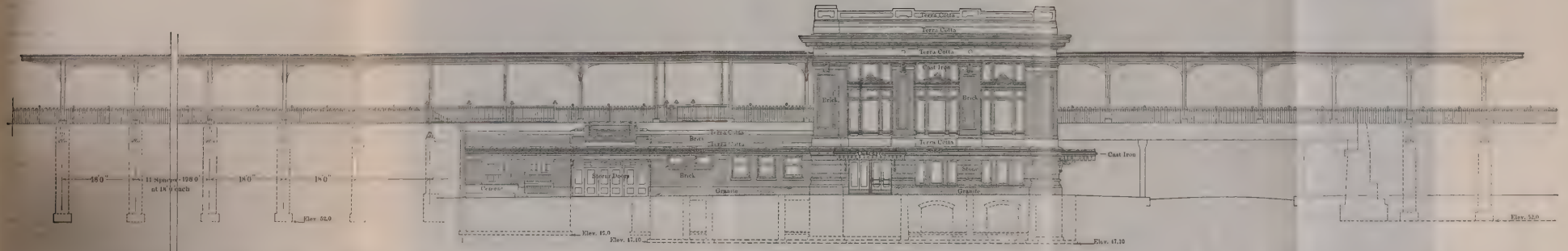
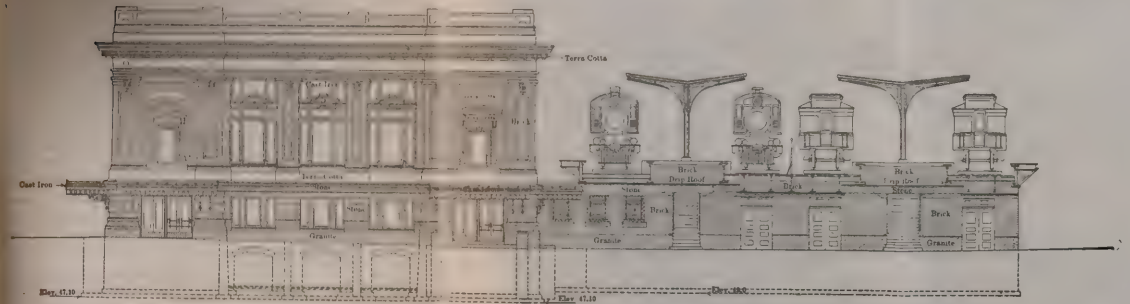
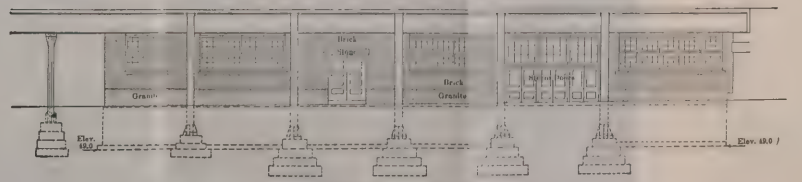


NINTH STREET ELEVATION

COLUMBIA AVENUE STATION



COLUMBIA AVENUE ELEVATION



HUTCHINSON STREET ELEVATION

There are two baggage elevators on each platform, and they are located so as to be accessible from the passageway previously referred to. They are of the hydraulic-plunger type, the pumps being operated by a 20-h.p. electric motor. The cars are 5 ft. 10 in. by 15 ft. 9 in., and the vertical lift is about 17 ft. 6 in. Each elevator has a capacity of 4 000 lb. in addition to its own weight, and travels at the rate of 100 ft. per min., with an average load of 2 500 lb. and with a maintained water pressure of 160 lb.

On the abandonment of the old station, a temporary building was erected on the street level at Oxford Street. This was used while travel was on the low level, and also when it was on the high level on the east side. Access to it at this stage was maintained by a stairway from Oxford Street and a temporary baggage lift between Oxford Street and Columbia Avenue. On the completion of the main waiting- and baggage-rooms, the temporary buildings were removed, and all travel was handled from the east platform until all four tracks were placed in service. The cost of the station building, platforms, shelters, etc., was $217 134.15.

Engine "Rocket".—Facing on the passageway to the trains, in the Columbia Avenue Station, there is an inclosed room, electrically lighted, in which the Engine "Rocket" rests on the original stone ties, cast-iron chairs, and wrought-iron rails which were recovered from the excavations made along the line of the elevated work, and which constituted the original roadbed of the Philadelphia, Germantown, and Norristown Railroad. A large number of the ties were found, a few of the chairs, and only a small quantity of the rails.

The "Rocket" was one of a number ordered by the Philadelphia and Reading Railroad in 1836 from Braithwaite and Millner, of London, England, and was built in 1838. It was delivered at the Port of Philadelphia, and conveyed by canal to the foot of Penn Street, Reading. From this point, it was hauled on its own wheels, through the streets, by horses, and placed on the tracks of the Philadelphia and Reading Railroad at Seventh and Penn Streets, Reading. The railroad was completed at that time between Reading and Norristown. It was placed in passenger service in May, 1838, and was retired in March, 1879, having run 310 164 miles. It was placed on exhibition at the World's Fair at Chicago in 1893, and also at the St. Louis Exposition in 1904. It weighs 8.4 tons.

Tioga and Nicetown.—These two stations may be referred to together, as they are in many respects similar. They are in a semi-suburban section of the City, and are used only for local passenger business. They are one-story buildings above the track level, with an additional story on the street level. The larger station at each point is on the south-bound side, in which is also the ticket office. They are built of brick with terra cotta trimmings, and are neatly finished throughout. The platforms can be reached by several flights of steps or by paths through the station grounds. Electric lifts are provided on both sides of the tracks for baggage and express service, and tunnels connect the north- and south-bound stations beneath the tracks. The elevators have a lifting capacity of 2 000 lb. at a speed of 50 ft. per min. The dimensions of the cars are 5 ft. by 10 ft.

The new station at Tioga, including the grading of the grounds, the cement platforms, and the driveways, cost $62 886.64. That at Nicetown, with similar appurtenances, cost $43 554.57.

While the two tracks on the west side were being elevated, the station buildings on this side were being erected, and when travel was placed on that side, a temporary platform was erected to care for the north-bound business, while the two tracks on the east side and the north-bound station were being completed.

Freight and Car Cleaning Yards.

The Railway Company operated freight yards at Eighth and Master, Tenth and Berks, and Broad and Lehigh Streets, before the elevated work was started, as well as car cleaning yards at Twelfth and York and Thirteenth and Lehigh Streets. The finished general plans provided for the retention of the first two freight yards and the rearrangement of the other so as to consolidate the car cleaning yards at Broad and Lehigh Streets. The yards between York and Cumberland Streets were designed for freight purposes. All were designed as elevated yards except that for the car cleaning, in which there was but slight change in grade. A number of studies were made to determine the best and most economical method of arranging the grades of the tracks in these yards, as well as the grades of the driveway approaches; and the final plans, it is believed, embody the most economical arrangement that could be made. The yards in all cases were thrown entirely out of service during construction. Concrete walls were built on the

PLATE LXXIII.
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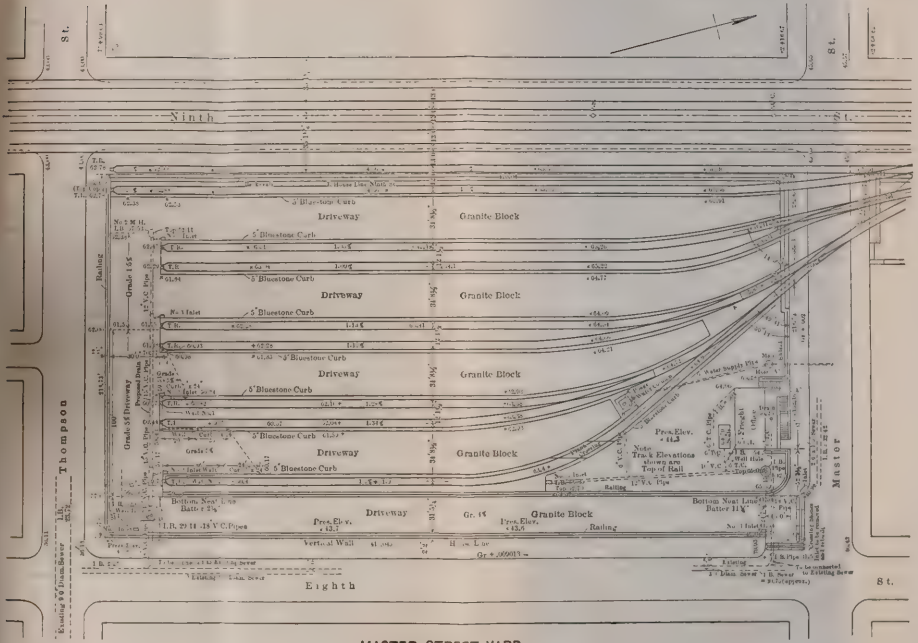
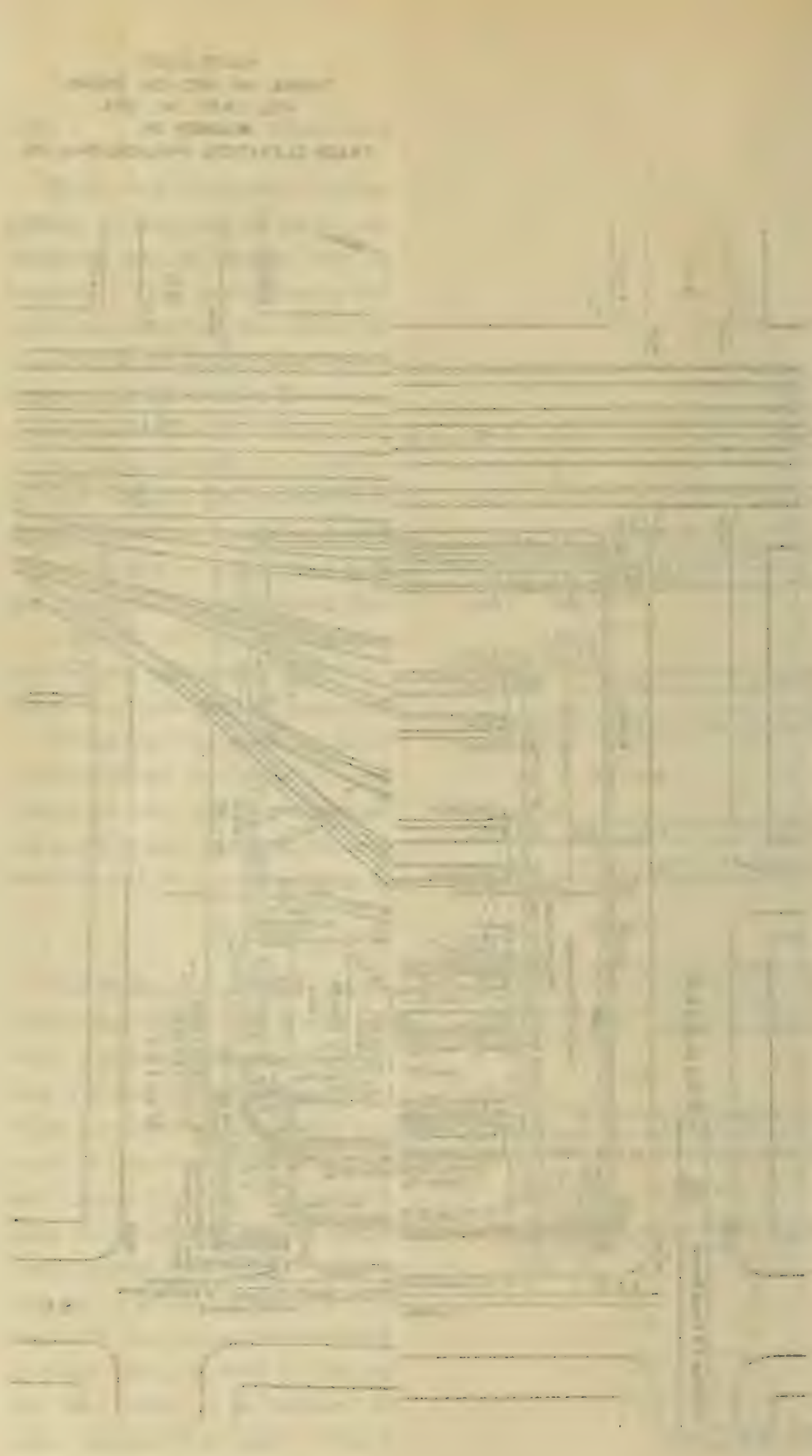
1874A



MASTER STREET YARD
GENERAL PLAN



street lines, the embankments placed to the new level, and all driveways provided with 5-in. bluestone curbs, and paved with granite blocks on a sand base. As a general rule, all ramp driveways leading to the streets had 4% grades, and the maximum grade on the tracks was 3 per cent. All track work was done by Company forces on emergency orders. The contracts for the freight yards were as follows, and show all the cost except the track work:

Location.	Total cost.	Car capacity
Master Street	$97 868.30	66
Berks Street	70 350.03	55
York Street Yards	67 059.23	104

Car Cleaning Yard.—This yard has a capacity of 230 passenger cars, and is provided with a service building, a small car repair and paint shop, a power-house building for furnishing steam, air, and lighting, and the yard is equipped with complete piping for the delivery of steam, air, and water to any point. All pipes are placed in creosoted wooden conduits, readily accessible for necessary repairs. The mechanical equipment of the power-house was furnished by the Motive Power Department, and its cost is not included. The entire cost, exclusive of the track work and the mechanical equipment, was $53 326.82.

Green Street Engine Yard.

Practically all the engines operating in the Reading Terminal are cared for in the Green Street Yard. The four main tracks and a track for the delivery of coal to the coaling station run through this yard on an elevated structure of concrete walls and fill. Engines coming from the Terminal run north to about Brown Street and back down a 3% grade on the east side of the yard to the original level beneath the coaling station, where they receive coal, water, and sand, and dump ashes. They then run south, turning if necessary on a 75-ft. turn-table operated by an electric motor, and pass through a tunnel beneath the main tracks to the west side of the yard where they can proceed to the engine-houses for repairs and cleaning, or up a second 3% grade to the high-level storage yard from which they run out on the main tracks and back to the Terminal. Thus all crossing of the main tracks is avoided. There are two turn-tables on the west side of the yard. This arrangement makes four bridges over Fairmount Avenue at different levels. Beginning on the west side, one track for engines leaving

the yard; four main tracks; one coaling track for the coaling station; and one track for engines entering the yard. This arrangement made it unnecessary to raise the whole of the yard, with the engine-houses and turn-table on the west side, to the new elevation, and thus effected a material saving in the cost of the work.

Coal-Pocket Yards.

The Railway Company owns and leases several large coal yards, one on the east side of the tracks between Berks and Norris Streets, and one on the west side at Tenth and Norris Streets. Only the first will be described, the other being similar but smaller.

Originally, the tracks in the yards were about 15 ft. above the street level. Due to the elevation of the main tracks, the difference in level was increased to about 41 ft., and it was decided to construct pocket yards, on account of the great height. There are two double-track pockets, parallel to the main tracks, each with a capacity of 10 000 tons in the pockets. Each is about 46 ft. wide and 500 ft. long. The two pockets are separated by a driveway about 37 ft. wide. The bottoms of the pockets are about 10 ft. above the paved surface of the yards, thus allowing delivery teams to be driven under them and loaded through chutes.

The type of construction was determined only after very careful study. Designs were made for all steel, reinforced concrete, and all wood, and the costs were carefully estimated; a number of different combinations were also tried. It was found, finally, that the composite design finally constructed was cheaper, both in first and capitalized cost, than either all steel or reinforced concrete, and as cheap in first cost as all wood. It consists essentially of a covered double-track steel plate-girder viaduct, wooden longitudinal pocket walls, and wooden pocket floors, supported on vertical concrete transverse partition walls of full height, without reinforcement. Each complete pocket is divided into 20 panels by the concrete walls, which are placed at 25-ft. centers. These walls are 46 ft. long, about 3.5 ft. thick at the base, where they rest on footings of reinforced concrete 4 ft. deep, 10 ft. wide, and about 49 ft. long.

Coal is delivered to the pockets by standard-gauge cars on each side of the longitudinal partition, each track being supported by two lines of 30-in. steel plate girders, at 5-ft. centers. The tracks are enclosed by a monitor of frame and corrugated iron with a slag roof.

PLATE LXXIV.
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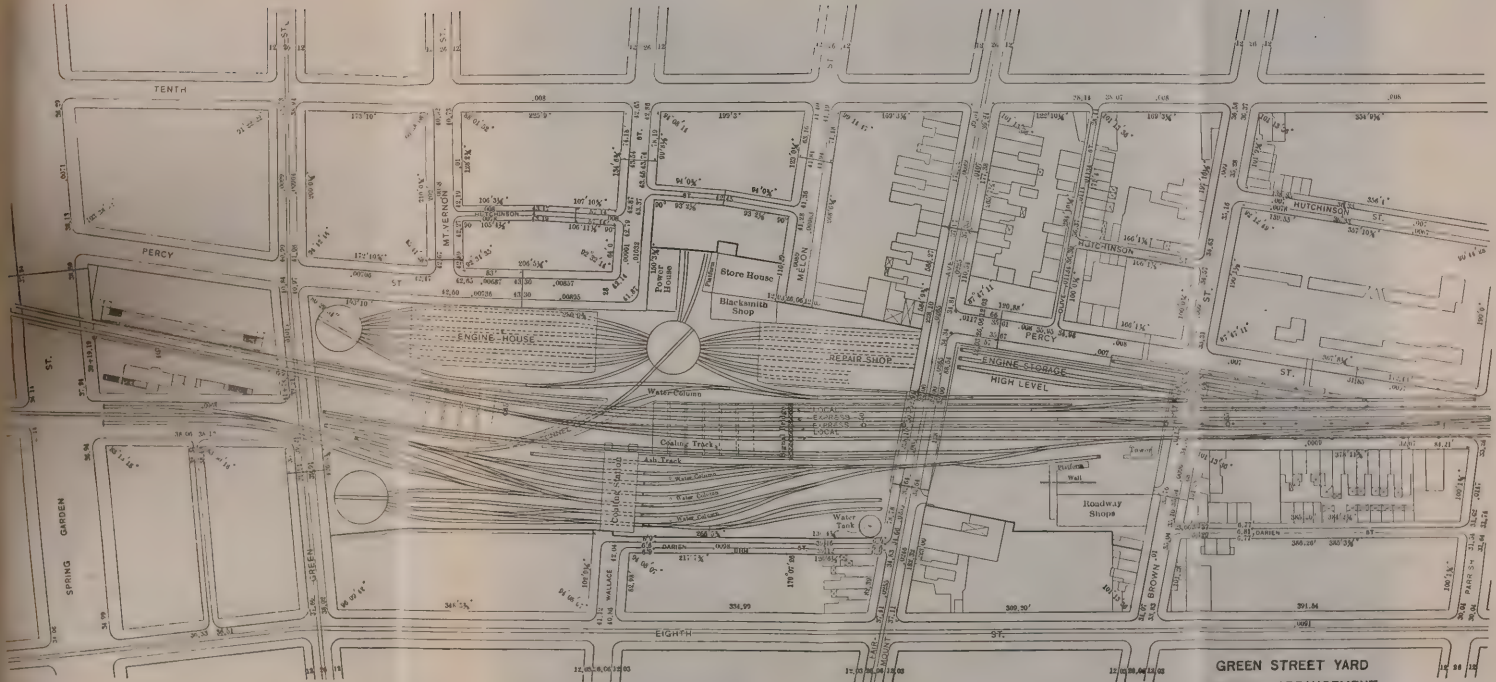

GREEN STREET YARD
GENERAL ARRANGEMENT

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There are about 5 000 cu. yd. of concrete, and 1 244 000 ft. b. m. of lumber in each line of pockets.

The cost of the yard covering the pockets, stables for 114 horses, paving, drainage, etc., was $273 578.29.

The yard at Tenth and Norris Streets, which is of similar construction, but with a total capacity of 1 170 tons, and stable room for 30 horses, cost $42 884.33.

Locomotive Coaling Station.

The original locomotive coaling station and ash-handling plant in the Green Street Yard consisted of a mechanical plant with a wrought-iron frame building supporting a steel-frame coal pocket with a storage capacity of about 400 tons, the coal being dumped into hoppers beneath the tracks and raised into the pocket by Hunt conveyors. It was constructed about 1894. The ashes were raised by a similar arrangement and loaded from small pockets into cars placed beneath. The metal in the framework was almost entirely destroyed by the lomocotive fumes, and the pocket was temporarily shored up when the work of elevating the tracks required its removal to the east side of the yard.

During the construction of the elevated work on the east side, requiring the entire abandonment of that portion of the engine yard, a temporary yard was built at Wayne Junction and kept in service until the new one was constructed.

Bids and detailed general plans were asked for the new station, based on a general plan of the pocket located at right angles to the main tracks and provided with ashes- and sand-handling facilities in addition to the handling of coal. The specifications called for the following essentials in the design to be submitted:

Building.—Foundations of concrete, and all pits below the surface of the ground to be connected with adjacent sewers, or a sump and pumps provided for caring for the drainage. From the level of the tracks up to and including the floor of the pockets, the construction to be of steel protected throughout with concrete; from the floor up, to be of steel protected with concrete, or reinforced concrete containing no exposed metal. The building to cover seven tracks, one for receiving ashes from a pocket above, and six for delivering coal to, or receiving ashes from, engines. Coal to be delivered from a special elevated track to the east of the main tracks, and, at the same level,

to be dumped into hoppers and raised into the pocket. The pockets to be twelve in number, two over each coaling track. The combined storage to be 2 000 tons. Each pocket to be provided with approved chutes, and with steam pipes and valves to thaw the coal when frozen. Ash pits to be arranged so as to enable engines to coal and dump ashes at the same time, and be supplied with water for quenching the ashes. Provision to be made for pockets for storing 250 cu. yd. of ashes. Hoppers with a storage of 8 cu. yd. of dry sand delivered from the high-level coaling track, and the necessary piping to transport the sand by air pressure to the delivery pockets, each of 4 cu. yd. capacity, one over each coaling track.

Machinery.—Duplicate machinery to be provided throughout, and designed for economy in operation and maintenance; also, to be as noiseless as possible. Each coal-handling unit to have a guaranteed capacity of 100 tons per hour. Machinery for handling ashes to have a capacity of 250 cu. yd. in 10 hours from the six coaling tracks.

Power.—All power to be electricity, which will be brought to the building by the Railway Company from a power-house on the west side of the yard.

A number of bids were received, based on different methods of handling both coal and ashes, and the contract was finally awarded to the Link Belt Company. The total cost of the work was $96 670.13. The principal items entering into this cost were as follows:

Building, drainage, etc.	$48 090
Coal-handling machinery	22 200
Ashes-handling machinery	8 590
Sand-handling machinery	2 270

The proposals required the bidders to name the following estimated figures, which in the accepted bid were as follows:

Cost of handling coal from cars to locomotive, based on 700 tons in 10 hours	$0.0225	per ton.
Repairs and maintenance on coal-handling machinery	0.007	" "
Cost of handling ashes from pits to cars, based on 250 cu. yd. per 10 hours	0.059	" cu. yd.
Repairs and maintenance, ashes-handling machinery	0.012	" " "

The general arrangement of the plant is shown on Plate LXXV.

1879

FIG. 27.—CUMBERLAND STREET BEFORE WORK WAS STARTED.

FIG. 28.—CUMBERLAND STREET AFTER WORK WAS COMPLETED.

1881

FIG. 29.—COALING STATION; AND EAST SIDE OF GREEN STREET YARD.

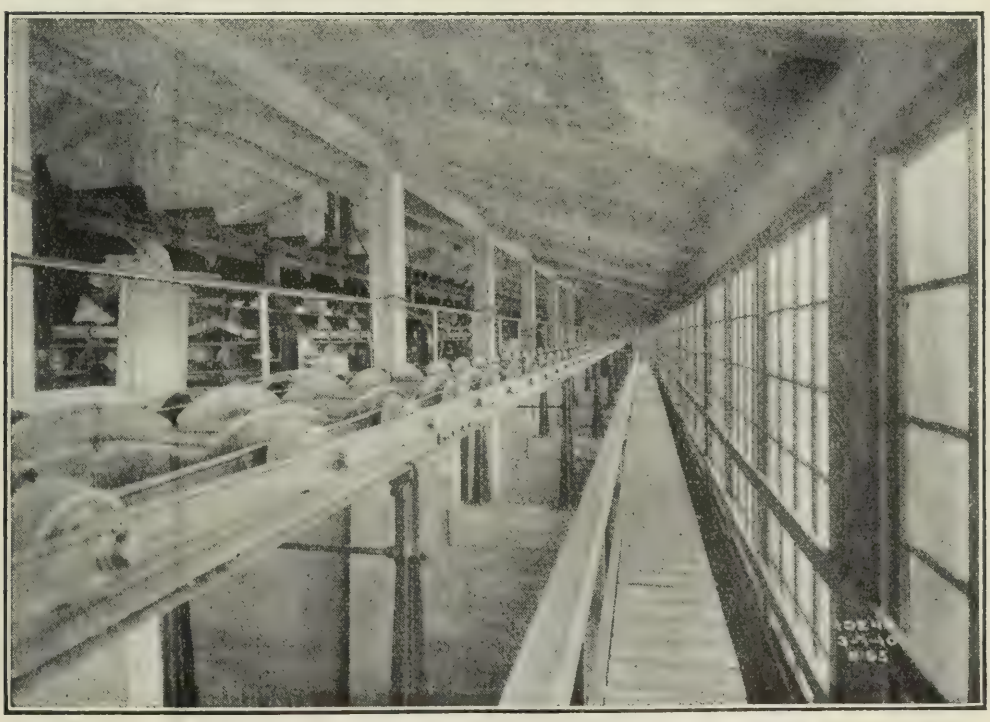

FIG. 30.—MACHINERY FLOOR, INTERIOR OF COALING STATION.

1884.

PLATE LXXV.
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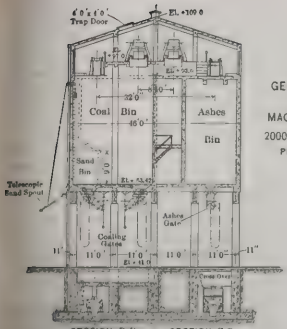
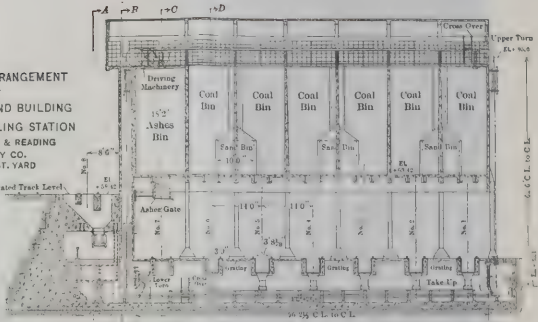
1898 A

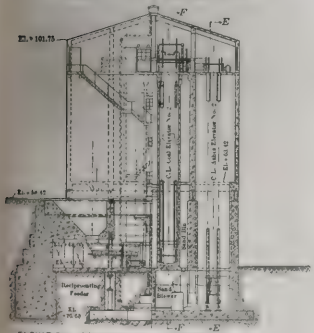
GENERAL ARRANGEMENT OF MACHINERY AND BUILDING 2000-TON COALING STATION PHILADELPHIA & READING RAILWAY CO. GREEN ST. YARD



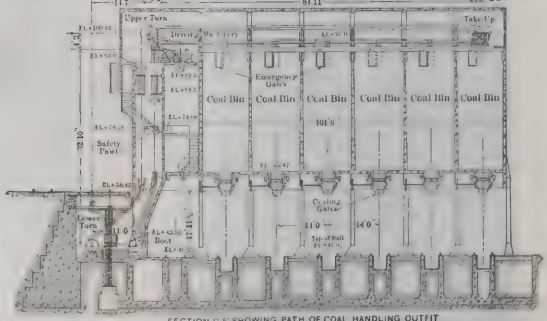
SECTION D-D SECTION C-C



SECTION E-E SHOWING PATH OF ASHES HANDLING OUTFIT



ELEVATION & SECTION A-A. SECTION B B.



SECTION F-F SHOWING PATH OF COAL HANDLING OUTFIT

The coal-handling machinery consisted of reciprocating feeders to two Link Belt, gravity-discharge, combined elevators and conveyors. On the horizontal runs the roller chains travel on steel tracks placed on the conveying runs so as to suspend the buckets above the cast-iron trough along which the coal is conveyed. Each machine has a chain speed of 75 ft. per min., and is actuated by a 25-h.p. General Electric motor, the motive power being located in the machinery room.

The fire-boxes of engines are cleaned into concrete brick-lined pits, and scraped through funnel-shaped cast-iron hoppers, with duplex gates and operating valves, to duplex-pivoted overlapping bucket carriers following rectangular paths and discharging into the storage pockets. These buckets are overlapping and of malleable iron, pivotally secured to two strands of heavy malleable-iron roller chain.

Dry sand is delivered from the receiving hopper, through pipes with reinforced bends, under an air pressure of 70 lb. Special chutes lead from the delivery hoppers to the engines, supplying sand by gravity.

The entire foundation of the pockets is of reinforced concrete, and this material is used for the construction above the floor of the pockets, and is properly designed according to the requirements of the Bureau of Building Inspection, City of Philadelphia. The drainage is arranged so as to lead directly to the city sewers.

The high-level delivery track is constructed on a 1% grade, the station being located midway in its length, so that ten loaded coal cars can be placed at the upper end of the grade, dropped by gravity to the hoppers, there unloaded, and dropped out of the way, without reaching the main tracks.

Permanent Tracks.

The main running tracks, previous to starting work, were laid with rails of 90-lb. Am. Soc. C. E. Section, the side tracks and yards with lighter and varying sections. A complete survey and joint inventory was made of all the existing track materials, by representatives of the City and the Railway Company, before the work was started. This was for the purpose of eventually determining the respective shares of each in the cost of the permanent tracks, as the number of tracks on the completed work was in excess of the original number, and in many cases the new tracks were laid with heavier rails and fittings. The City only joined in the cost of replacing the original weight, all

excess being a betterment to the Railway Company and paid for entirely by them. All track details were furnished according to the standard plans and specifications of the Railway Company.

The most interesting details of these specifications are given herewith:

Rails.—90-lb. Am. Soc. C. E. Section. Shrinkage, 6.5 in. in 33 ft. Chemical composition:

Carbon ..... 0.55 to 0.65%, averaging 0.58%
Phosphorus ..... not to exceed 0.075%
Silicon ..... not to exceed 0.20%
Manganese.....0.90 to 1.15%, with average above 1.00%

Drop test: 3 ft. between centers of supports; tup, 2 000 lb.; anvil block, 20 000 lb.; striking face of tup, not more than 5 in.; height of drop, 18 ft.

Ties.—First-class ties of white or rock oak. Dimensions, 8 ft. 6 in. long, 7 in. thick, and not less than 7 in. nor more than 14 in. on the narrowest face.

Ballast.—Good, hard, durable, broken trap rock, crushed so that it will not be larger than will pass through a 2.5-in. ring.

Specials.—100% joint-plates; Verona nut-locks; hard-steel, centered frogs; Wharton guard-rail clamps; Ramapo or Pennsylvania Steel Company switch stands; O'Brien insulated joints.

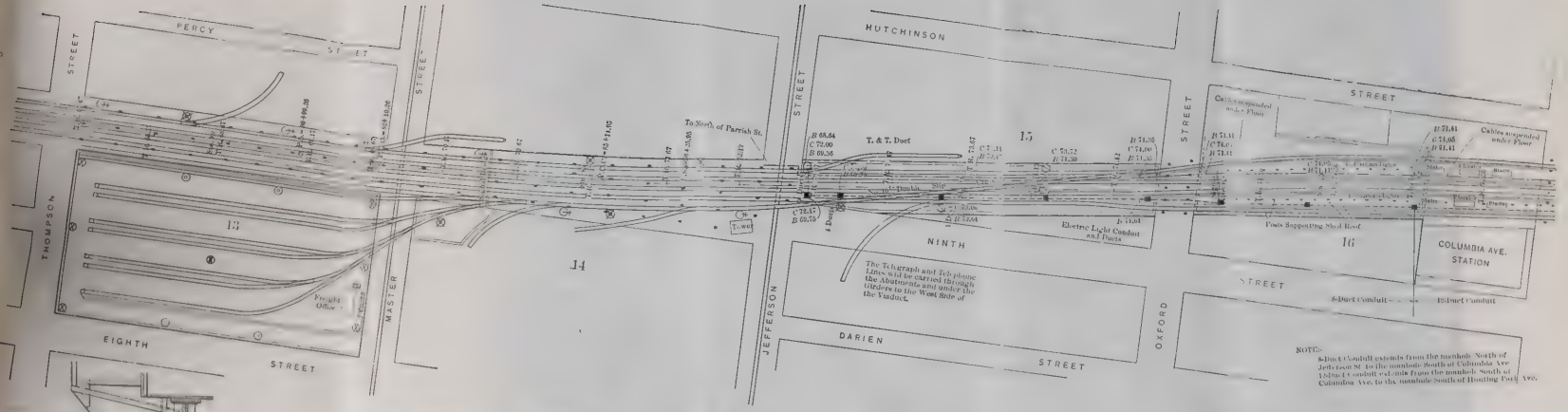
Temporary Signals.

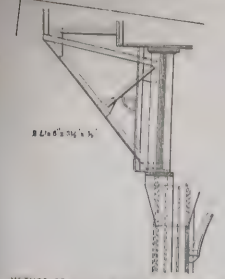
During the changes in the tracks, temporary block signals were erected and maintained throughout the entire length of the line, using old material, as far as possible, and adapting it temporarily to the new conditions. Where lead-outs to the important yards were maintained, or where it was necessary to pass from a two-track to a four-track condition, temporary towers were erected and connected with the necessary switches and signals with manual interlocking. All this was done by Company forces, on emergency orders covering the temporary track work, thus keeping the operation of trains at all times under the control of the Operating Department, and avoiding all possibility of interference with the safe movement of trains.

Permanent Signals.

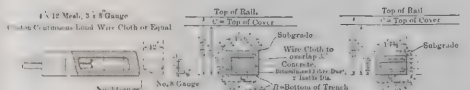
Before the work of elevation was started, the entire line was equipped with Hall automatic block signals, and two small manual

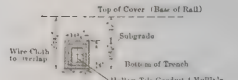
1894.


PERCY STREET
THOMPSON STREET
MASTER STREET
JEFFERSON STREET
HUTCHINSON STREET
OXFORD STREET
EIGHTH STREET
NINTH STREET
DARIEN STREET
COLUMBIA AVE. STATION
To North of Parrish St.
T. & T. Duct
Double Slip
Electric Light Conduit and Ducts
Posts Supporting Shed Roof
Cables suspended under Floor
Freight Office
The Telegraph and Telephone Lines shall be carried through the Abutments and under the Girders to the West Side of the Viaduct.
NOTES:
8-Duct Conduit extends from the manhole North of Jefferson St. to the manhole South of Columbia Ave.
15-Duct Conduit extends from the manhole South of Columbia Ave. to the manhole South of Hunting Park Ave.
8-Duct Conduit
15-Duct Conduit



METHOD OF CARRYING CABLE ON SIDE OF VIADUCT



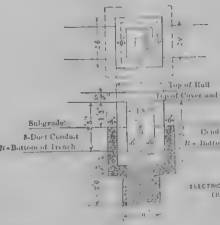


TELEGRAPH AND TELEPHONE DUCT.

Pole Symbols

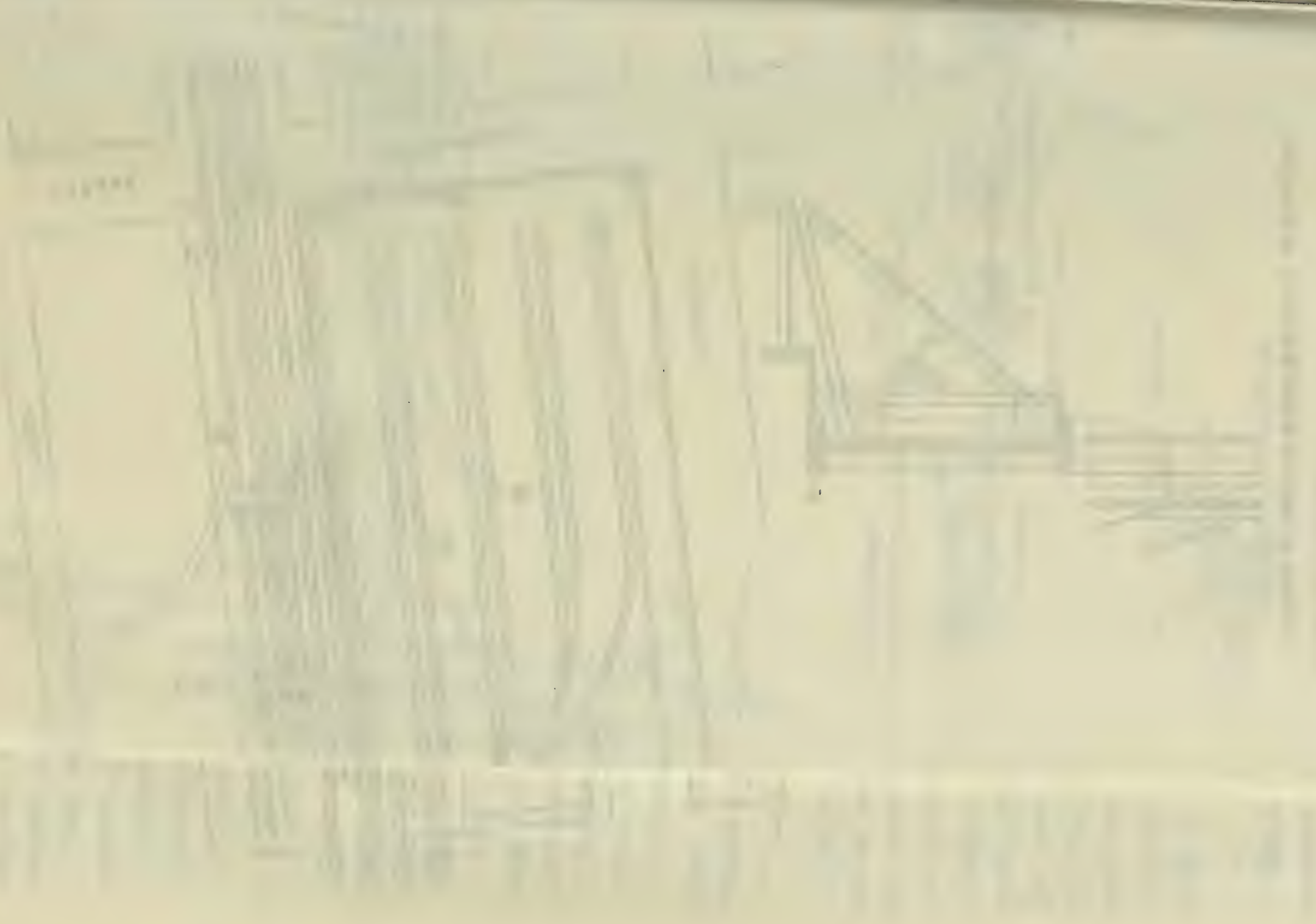
- Existing Wooden Poles
- New Wooden Line Poles.
- New Wooden Light Poles.
- Bracket Lights.
- Iron Line Poles.
- Iron Line and Light Poles.
- Iron Light Poles (4 Tracks)
- Iron Light Poles (5 Tracks)
- Manhole

All Ducts (Loricated or Bituminized) to have an Inside Diameter of 2 Inches.



WORKING PLAN SHOWING

PERMANENT ELECTRIC LIGHT AND TELEGRAPH AND TELEPHONE LINES.



interlocking plants were located at Fairmount Avenue and Huntingdon Street, respectively, to control the shifting into the yards at these points. After the elevated track plans were completed, it was decided to operate all the switches between Broad and Green Streets affecting the four main tracks from electro-pneumatic interlocking plants, aggregating 98 working levers. These are as follows:

Cumberland Street	11	working levers.
Tenth Street	39	" "
Jefferson Street	21	" "
Brown Street	27	" "
	98	working levers.

The new signaling apparatus was put in by the Union Switch and Signal Company, and embodied the latest practice, such as the following:

1. The auxiliary control of all signals by all switches by means of the duplications of positions of both switches and signals with relays;
2. Approach and route locking;
3. Detector circuits;
4. Federal switch guards.

The high signals are semi-automatic, three-position, lower-quadrant type, with a short "all route" or "call-on" signal beneath, on the same mast. All these signal masts are carried on signal bridges.

The short-arm signal is a two-position signal—horizontal for "stop" and 45° below the horizontal for "caution—proceed." It is used to call trains on past the high signal or signals on the same mast, if, for any reason, the proper high signal cannot be displayed in the "proceed" position, and this "call-on" feature is accomplished by a push-button placed directly under the signal lever. In addition to its "call-on" functions, the short-arm signal governs all the remaining routes, other than the two most important ones controlled by the two high signals above it. The maximum number of high arms on any one mast is two, governing only one route each.

The Federal switch guards are 55 ft. in length, and are placed on every switch, movable-point frog, derail, etc., over the entire territory. The detector circuits are an average of 130 ft. in length.

The route-locking on the various plants is arranged with a releasing feature, whereby all switches in the rear of a train on the route are released as fast as the entire train clears the respective fouling limits of the several switches involved in the route. Thus a route cannot be changed ahead of a train, but may be changed behind one, provided the home signal has been put to the "stop" position. Hand time releases are installed for the release, when necessary, of signal levers affected by the approach locking feature.

Electric lights (incandescent) of ¼-c.p. are applied to all switch levers, this being supplementary to the detector circuit locking. Each light is encased in an iron box with a separate iron cover in which there is a frosted-glass disc. This iron box is directly under the levers, and is at all times in view of the signalman. These indicating lights are normally lighted up, illuminating the small white frosted-glass disc with number thereon corresponding to its respective switch lever number, thus showing the signalman that there is no train on the track circuit governing the switch, and the lever is free to be thrown. The extinction of the light indicates the presence of a train on the switch, and that lever is locked and cannot be thrown.

Approach and home indicators are provided in the tower. The approach indicator is of the disc type, and the home indicator, indicating from the home signal entirely through the interlocking, is of the semaphore type.

The semi-automatic high semaphore signals at the entrance to the interlocking provide the signal aspect for automatic indication through the interlocking block, that is, these signals will assume the "proceed—caution" (45°) position when the interlocking block is unoccupied, and when the track and switches are right for the route governed, and when the signalman throws the signal lever, that is, these signals will go on from the "caution" to the "proceed full speed" (90°) position, if conditions are right for two blocks ahead. They will go automatically to the stop position, after the train has passed them and entered the interlocking block.

The automatic signal aspect at the end of the interlocking block—governing the territory between and up to the next interlocking plant—is a double-head "home" and "distant," normal danger, Hall disc signal. These disc signals are wired in conjunction with the semi-automatic semaphore signals, so that, in all cases, for all kinds of signals, the

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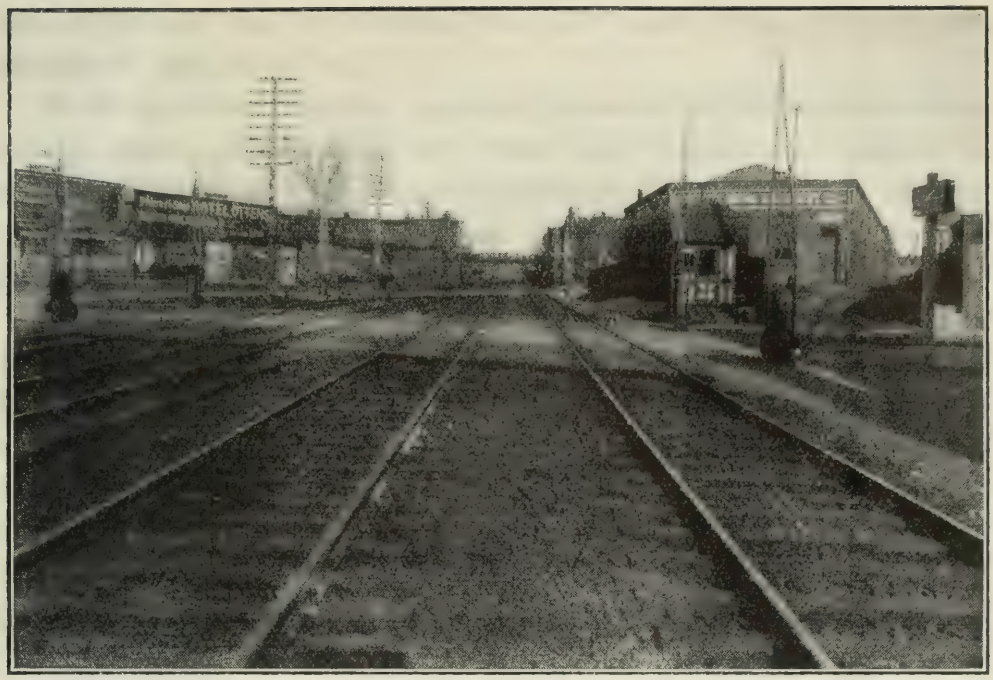


FIG. 31.—WESTMORELAND AND NINETEENTH STREETS BEFORE WORK WAS STARTED.

FIG. 32.—WESTMORELAND AND NINETEENTH STREETS AFTER WORK WAS COMPLETED.

proper home and distant indications are given, and also so that the disc signals through the interlocking will only clear if the proper route, leading to them, is set up.

At Broad Street the new signaling connects up with three-position semi-automatic signals, and with home and distant disc signals, as above explained. The signaling at the Green Street end joins up with two-position normal clear automatic pneumatic semaphore home and distant signals, governing into the Reading Terminal.

Eight iron signal bridges on the elevated carry the signal masts for the high semi-automatic semaphore signals, short-arm signals, and side signals. Table 7 gives the location, number of tracks crossed, and cost of same. At Columbia Avenue Station, where the tracks spread apart on account of the station platforms, there is a two-track signal bridge north of the station carrying signals for the south-bound tracks, and a two-track bridge south of the station with the signals for the north-bound movement. In all cases the clearance from underside of signal bridge to top of rail is 23 ft. 8 in.; and the side clearance from leg of bridge to center line of nearest track is an average of 7 ft.

Throughout the entire territory, between Broad and Green Streets, all switches are interlocked and controlled by machine.

TABLE 7.

Contract No.	Location.	Number of tracks.	Cost.
47	South of Fairmount Avenue	5	$1 684
	North of Poplar Street	4	1 558
	North of Master Street	4	1 444
	North of Oxford Street—two bridges	4	1 489
	North of Columbia Avenue	5	1 595
33	North of Norris Street	8	1 503
	Twelfth and York Streets	5	1 100
	Thirteenth and Cumberland Streets	5	1 174

Power plants, at Huntingdon and Wallace Streets, supply the compressed air (100 lb. per sq. in. at the power-house) to the towers through a 2-in. main air pipe, which is buried 6 in. in the ballast alongside the track. There are reservoirs in the main air pipe for drainage, and also auxiliary reservoirs in the $\frac{3}{4}$-in. branch pipes between the functions operated and the main air pipe.

Electric current is furnished from the power plant at Huntingdon Street at 650 volts and 25 amperes (D. C.) for charging the necessary storage batteries at the various interlocking towers.

The charging wire, etc., and all signal wires and cables, are laid in trunking on top, at the side, and between the tracks of the elevated structure. There is a main switch-board and rheostat at the power house at Huntingdon Street controlling the current for the charging of the storage batteries, and also a switch-board at each plant for the control of charging and discharging. These switch-boards at the several plants are wired so that, if desired, all the batteries at all four plants can be charged in a series at one time, or, on the other hand, any battery or batteries desired can be cut off from charge and put on discharge.

Electricity is used for the lighting of all signals, towers, etc. The signal lanterns are specially designed, each containing two 2-c.p. incandescent electric lights, and, for emergency use, an oil fount, wick, etc. The high-tension (1 100 volts, A. C.) wire for the electric lighting is carried in a separate conduit from the trunking used for the signal wires, and this current, by means of transformers on the ground at each signal group, is stepped down and changed to 110 volts (D. C.) for the signal lamps. Separate trunking is also run for these 110-volt wires.

Communication between the towers is provided for by train describer and telephones. These train describers consist of sending and receiving instruments for each track, and the description of traffic is accomplished by the signalman pressing certain numbered buttons (according to code) on the sending instrument for a certain track. This action appears on a corresponding receiving instrument at the next tower, giving full information to this signalman of coming train movement.

The signal tower at Cumberland Street is set back from the center line of the track 10 ft., and is a brick building, three stories from the street—two stories above the elevated structure. It has brick facings and an overhanging tile roof. The inside dimensions are 21 ft. 9 in. by 11 ft. 8 in. The machine is in the top story; on the track level are the relay racks and relays and the storage batteries. On the street level the room is used for the signal maintainer and for the furnace.

The tower at Tenth Street is the largest of the four. It is triangular in plan, with the hypotenuse running parallel with the railroad. The arrangement of the rooms is similar to that at Cumberland Street.

The tower at Jefferson Street is built on the viaduct and back

10 ft. from the center line of the track. It is a copper-sheathed, steel structure, with overhanging tile roof. The inside dimensions are 26 ft. by 11 ft. It is two stories high.

The tower at Brown Street is set back from the center line of the track 9 ft. It is of brick, two stories high, and similar to that at Cumberland Street. The inside dimensions are 25 ft. 10 in. by 11 ft. 3 in. In this tower is placed a temporary six-lever frame machine, with four working levers to take care of the converging of the new four-track line into the old two-track line at Green Street. This will be removed when the four-tracking is completed into the Reading Terminal.

The cost of the signal towers was as follows:

Location.	Cost.
Cumberland Street	$3 990
Tenth Street	5 990
Jefferson Street	4 114
Brown Street	3 760

The entire cost of the interlocking work, exclusive of the signal towers and bridges, was $127 462.90.

Locomotive Water Supply.

On account of the changed conditions in the Green Street Engine Yard, it was necessary to lay new pipe lines, making connections with the existing water tanks on the west side of the yard; furnish and erect three new 12-in. Poage automatic water columns; re-erect an old 8-in. column on the west of the main tracks; and construct the pits and drainage to the city sewers. The total cost of this work was $6 107.27.

Changing Grades of City Streets.

All the plans for the necessary changes to the city streets were prepared by the Bureau of Surveys and approved by the Railway Company. The work was divided into five contracts.

The work under these contracts included the underpinning of the adjoining buildings, where the changes in the streets affected their foundations; the excavation between house lines to the new street level; the renewal and reconstruction of all gas and water mains, electrical conduits, and other underground structures; provision for temporary access to all buildings, and the repaving of driveways and

sidewalks. It also covered the necessary adjustments and repaving of streets, where there was no change of grade, but where the previous existence of the tracks of the railroad and their removal made such adjustments necessary. It also covered the construction of the smaller sewers, the work on which could be deferred until the streets were excavated. The work covered was generally as shown in Table 8.

TABLE 8.

Contract No.	Description.	Cost.
108.....	Changes of grade, Fairmount Avenue and other adjoining streets; paving Ninth Street from Brown to Jefferson Streets; changes of grade on Columbia Avenue and adjoining streets	$122 833.27
109.....	Adjustments of streets, Berks to York Streets; changes of grade of Cumberland, Thirteenth and adjoining streets.....	110 756.42
110.....	Changes of grade on Allegheny Avenue, Westmoreland, Ontario and adjoining streets affected	134 338.60
111.....	Changes of grade of Hunting Park Avenue and adjustments of all streets from Tioga Street to Richmond Branch.....	43 125.00
115.....	Adjustments on Montgomery Street	2 185.75
	Total.....	$413 239.04

The principal prices and the quantities of the most important items are given in Table 9.

TABLE 9—PRICES AND QUANTITIES OF PRINCIPAL ITEMS OF STREET CONTRACTS FOR GRADING AND PAVING.

Kind of Work.	Contract No. 108.		Contract No. 109.		Contract No. 110.		Contract No. 111.	
	Rate.	Amount.	Rate.	Amount.	Rate.	Amount.	Rate.	Amount.
Excavation, cubic yards..	$0.54	24 050	$0.45	30 547	$0.54	64 125	$0.54	20 918
Rubble underpinning, cubic yards.....	14.00	44	20.00	358	14.00	1 460	14.00	36
Granite block paving, square yards.....	3.15	3 676	2.90	2 350	3.15	309	3.15	1 066
Sheet asphalt paving, square yards.....	1.37	17 281	1.85	9 478	1.37	9 014	1.37	7 144
Cement sidewalk paving, square yards.....	1.19	12 815	1.25	8 782	1.19	6 329	1.19	3 194
6-in. cement curb, linear feet.....	0.75	1 097	1.50	3 837	0.75	1 032	0.75	.....
8-in. cement curb, linear feet.....	0.80	8 940	1.50	3 609	0.80	1 437	0.80	1 959
Granite block repaving, sand base, square yards.	0.45	1 964	0.40	1 410	0.45	5 497	0.45	2 324
Granite block paving, 6-in. concrete base, square yards.....	0.85	2 788	1.15	1 351	0.85	7 528	0.85	12

INDUSTRIAL CONNECTIONS.

Before the work was begun, there were many industrial connections to the existing tracks, leading to coal yards of retail dealers, to factories,

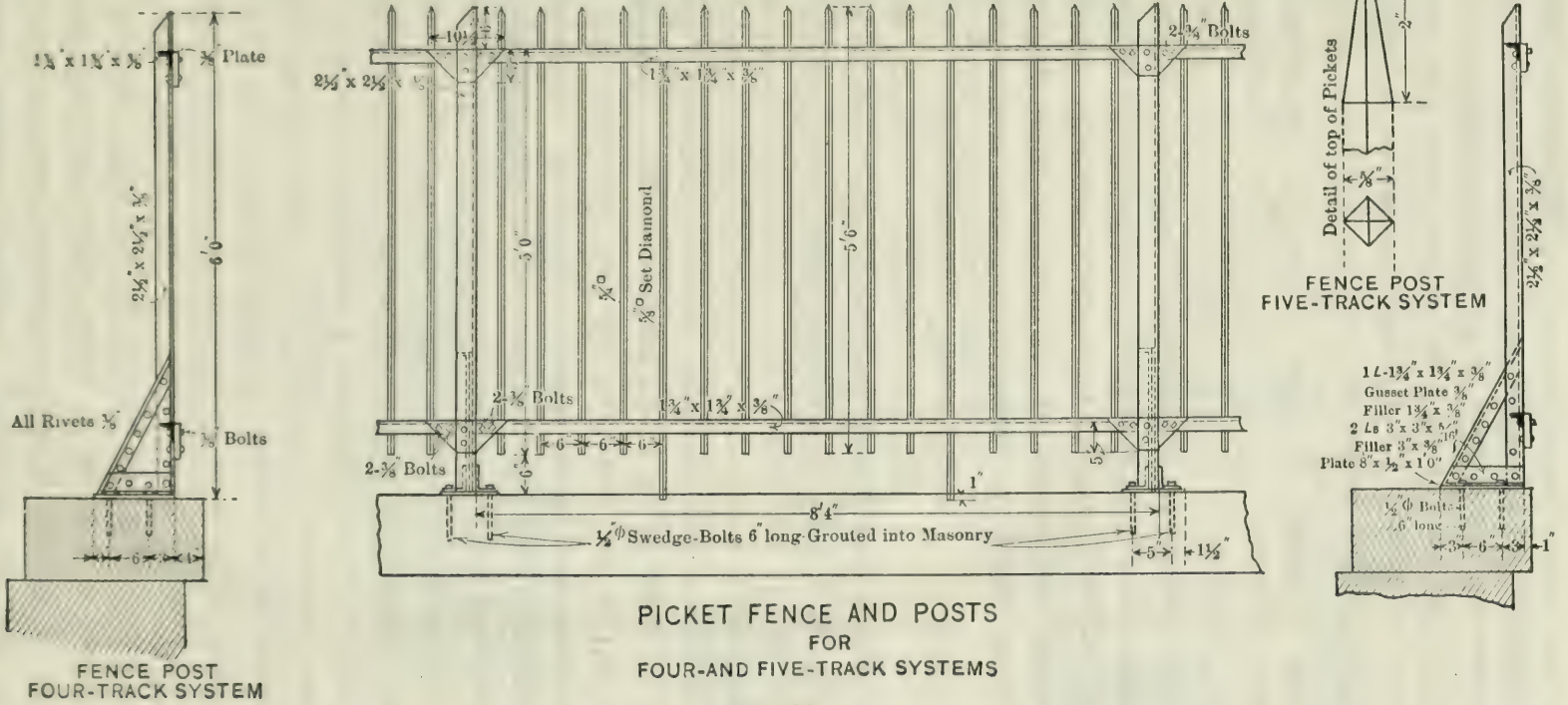
and for general industrial purposes. Under the law, the burden of the reconstruction of these connections falls principally on the owner, as the Railway Company cannot assist in the cost of reconstruction on private property. The cost of making the connections from the main tracks to the right of way was borne jointly by the City and the Railway Company as part of the work of elevation of the tracks, where industrial connections formerly existed. Owing to the requirements of the safety appliance clauses of the Interstate Commerce Commission, the Railway Company established a minimum radius of 150 ft. for the new connections, and, further, decided on a minimum length of siding to accommodate two cars. The radii of many of the former sidings was as small as 50 ft., in many cases requiring the insertion of special extra links between cars in operating, and even then attended by frequent derailments. In minimum cases, in the new work, a No. 4 curved frog was used, and, with the minimum radius, this made it possible to maintain many connections where the frontage on the railroad was small. However, the more important connections have been maintained, although suits are pending where the new conditions have made their continuance impossible. Where coal yard sidings have been reconnected, the additional height has made it possible for the owners in many cases to build pockets and load directly into wagons without extra handling.

Railings and Picket Fence.

Careful studies were made for the design of railings to be placed along the retaining walls, and finally the gas-pipe railing with cast-iron posts was adopted. The specifications for the pipe required that it be dipped at the shops in red lead paint containing 95% of red oxide of lead. The cost of this type of railing, including the attachment to the walls, was as follows:

Contract numbers...	5	7	14	32
Cost per linear foot..	$0.58	$0.75	$0.58	$0.68

After the work was well under way, the management of the Railway Company decided to enclose its property throughout, so as to prevent trespassing, and plans were prepared for a picket fence (Fig. 33), designed so as to make it very difficult to scale and at the same time be as economical as possible. Gates of the same type were also required at all the industrial connections.



PICKET FENCE AND POSTS
FOR
FOUR-AND FIVE-TRACK SYSTEMS

Fig. 33.

All the gas-pipe railing which had been furnished was stored and used on other contracts where the walls required merely protection for railway operatives, and were not on property lines.

Most of the picket fence was furnished under Contract No. 36, but prices were obtained in a number of other contracts. These prices included the priming with red lead and the erection and painting with two coats of standard Philadelphia and Reading paint. The prices were as follows:

Contract numbers ...	3	4	6	9	10	12	36
Fence per linear foot.	$1.21	$1.15	$0.90	$1.15	$1.15	$1.50	$1.05

The work under Contract No. 36 included a number of iron gates for the entrances of the various freight yards, an item of $2 580 for raising the Huntingdon Street foot bridge, and 8 819 ft. of picket fence.

The result of placing the picket fences has been a complete shutting out of trespassers from the tracks, as the only approach to them is from the freight yards or stations.

Telegraph and Telephone.

Before the elevation of the tracks, the telegraph and telephone system was carried on pole lines throughout the entire extent of the work. In addition to the wires of the Railway Company, those of the Western Union Telegraph Company (operated under a lease) had to be cared for.

As overhead wires during construction would have required constant changing, and would have been much in the way, it was decided to remove all through wires entirely from the line. Accordingly, arrangements were made to carry them in cables through city conduits to Thirty-first Street and Girard Avenue, and thence in a temporary cable attached to the existing pole line along the main line to West Falls, and thence by the Richmond Branch to Wayne Junction. The wires to yards and stations required for operation were carried through the city streets, by leased attachments on poles of private corporations, to the several yards and stations, and in a few cases along the line during construction in underground temporary trunking. The Railway Company elected to place these wires underground permanently on the solid elevated structure, by cables in vitrified clay ducts surrounded with concrete and in cables attached to the steel viaduct between Brown and Jefferson Streets. As the City was not willing to do more than join in the cost of an overhead pole line, it was

decided to have the Railway Company arrange and pay for the permanent installation, the City to join only in the cost of replacing an overhead pole line. In crossing the bridges, the cables are either carried beneath the steelwork, passing through openings provided in the back walls or above the floor carried through the haunching against the girders through bituminized fiber conduits laid in the concrete. Concrete manholes with steel covers are located at such intervals as the drawing of the cables makes necessary. The laying of the ducts and the drawing of the cables was done by Company forces, after travel was on the tracks and as soon as the fill was sufficiently consolidated.

Permanent Electric Lighting.

The original lighting of the line and the stations was by no means complete. The stations and yards were lighted either by gas or electricity, provided by outside corporations. The lighting of the new work is complete, not only in the yards and at the stations, but also along the entire line, the current being furnished by the power-houses of the Company at Wallace and Huntingdon Streets. The wires are carried from the power-houses along the line in ducts of bituminized fiber, laid in concrete on the masonry and fill sections, and by overhead poles attached to the steel viaduct. As this work was an entire betterment, it was carried out by Company forces in conjunction with the Motive Power Department, and the entire cost borne by the Railway Company.

General Data.

Total number of grade crossings abolished	28
Total number of bridges, not including viaduct	28
Length of four-track viaduct, in feet	3 365
Square feet of bridges, including viaduct	352 400
Total weight of structural steel in bridges and viaduct, in tons	27 180
Number of signal bridges	11
Third-class masonry in retaining walls, in cubic yards	67 500
Concrete masonry in abutments and walls, in cubic yards	151 800
Barrels of cement used	231 800
Embankment, in cubic yards	430 400
Lumber in temporary trestle, in thousands of feet	4 322
Number of contracts	47
Number of emergency orders	52

Station and other buildings	16
Miles of single track, permanent	30.1
Number of reconstructed industrial connections	46
Miles of telegraph and telephone conduit	3.8
Miles of electric light conduit	3.2
Excavation for changes in street grades, in cubic yards	139 620
Total length of sewers, in feet	17 835
Street driveway paving, all kinds, in square yards	73 192
Street sidewalk paving, all kinds, in square yards	31 120
Plans prepared by Railway and approved by City	347
Plans prepared by City and approved by Railway	50
Total number of plans prepared by Railway Company	3 200

PRINCIPAL CONSTRUCTION DATES.

July 6th, 1907. Work begun on Engineer's office, Huntingdon Street.

October 2d, 1907. Work begun on temporary freight yard, Nineteenth and Indiana.

August , 1907. Work begun on sewer contracts.

Between Berks and Broad Streets.

November , 1907. Work begun on temporary tracks, low level.

January 15th, 1908. Work begun on retaining walls, etc., east side.

January 30th, 1909. Two temporary tracks in service, high level, east side.

November 27th, 1909. Two permanent tracks in service, high level, west side.

January 4th, 1911. Schedule trains on all four tracks.

Between Seventeenth and Indiana and Richmond Branch.

October 4th, 1909. Work begun on walls, west side.

October 22d, 1910. Two temporary tracks in service, west side, high level.

June 4th, 1911. Full schedule travel over whole section.

Between Green and Berks Streets.

June 2d, 1909. Work begun on foundations for viaduct.

January 1st, 1911. Two tracks in service on east side, high level, on viaduct and trestle. All grade crossings gone.

December 17th, 1911. Full schedule travel on all tracks.

Acknowledgment.

The ordinance authorizing the work provided that it be carried out by the Chief Engineer of the Philadelphia and Reading Railway Company and the Director of the Department of Public Works of the City of Philadelphia. The original plans were signed on March 28th, 1907, by William Hunter, M. Am. Soc. C. E., as Chief Engineer of the Railway Company, and Mr. John R. Hathaway, as Director of the Department of Public Works; also by Theodore Voorhees, M. Am. Soc. C. E., Vice-President of the Railway Company, and George S. Webster, M. Am. Soc. C. E., Chief Engineer, Bureau of Surveys, City of Philadelphia. Since that time the same officials of the Railway Company have continued in office, while the Directors serving the City have been Messrs. George R. Stearns, Harry A. Mackey, and Morris L. Cooke. Mr. George F. Baer has been President of the Railway Company, and the Mayors of the City have been, the Hon. John Weaver, the Hon. John E. Reyburn, and the Hon. Rudolph Blankenburg.

The engineering work, in the preparation of the plans and specifications, as well as the work of construction, was carried out by the Railway Company under the direction of William Hunter, M. Am. Soc. C. E., as Chief Engineer, with the writer and Messrs. E. Chamberlain and Charles H. Hitchcock as Assistant Engineers; and by the City under the direction of George S. Webster, M. Am. Soc. C. E., as Chief Engineer, Bureau of Surveys, by Mr. James W. Phillips as Assistant Engineer in charge, and Assistant Engineers D. Jones Lucas, M. Am. Soc. C. E., Silas G. Griffith, H. T. Shelley, Arthur G. Singer, and Charles H. Stevens.

During the entire time, all bridge plans were passed upon by W. B. Riegner, M. Am. Soc. C. E., Engineer of Bridges, Philadelphia and Reading Railway, and George E. Datesman, M. Am. Soc. C. E., was Principal Assistant Engineer, Bureau of Surveys.

The Operating Department of the Railway Company rendered most valuable assistance throughout the entire work. Mr. Agnew T. Dice, General Manager, and the following officers of the Philadelphia Division, J. B. Warrington, Superintendent; H. C. Smith, M. Am. Soc. C. E., Division Engineer; Geo. W. Snook, Supervisor; and J. E. Peters and H. Mansfield, Trainmasters.

To Mr. A. H. Yocum, Signal Engineer, and to Mr. David S. Gendell,

Engineer of Erection, McClintic-Marshall Construction Company, the writer desires to express his appreciation for assistance in the preparation of details of this paper.

To all these gentlemen, the writer begs to acknowledge kind assistance, advice, and courtesy rendered at all times, and to his superior officers for permission to present the data contained in this paper.

DISCUSSION

Mr. Snow. J. P. Snow, M. Am. Soc. C. E.—It will be of interest to learn from the author if the scheme for this work was decreed by some public board, as is customary in some of our States, or by agreement between the City and the Railroad; also, if all matters of detail, like handling telegraph wires, which it is stated the City would not help pay for, were settled on in the original agreement or decided on as the work proceeded; also, if the Railroad was compelled to pay one-half the total cost of widening streets, where that was done.

In connection with building the temporary trestle, would it not have been entirely safe to have used timber sills resting directly on the street surface, and much cheaper than to build concrete footings? Where there was any question of the bearing value of the ground, cross-planks could have been placed under the sills. Apparently, this trestle was built along the street, the surface of which must have been well compacted and probably much better able to sustain loads than the soil beneath. The load per square foot under a simple sill would not much exceed 5 tons per sq. ft., and, with 30-in. cross-planks under the sills, it would reduce to about 2 tons, which is a safe load on very ordinary soil.

For temporary work, it is good practice to use timber in contact with the ground, and oftentimes the top crust of soil is capable of carrying a greater concentrated load than the material below.

Mr. Lewis. E. W. Lewis, M. Am. Soc. C. E.—The speaker would like to ask Mr. Wagner how near the original estimate the total cost of the work has proved to be, if he is permitted to give the figures. Some railroad corporations are averse to giving out any figures of actual cost of improvements, and the speaker does not wish to place the author in an embarrassing position. The cost of extensive improvements, in a number of cases which have come under the speaker's observation, has largely exceeded the estimates, in some cases as much as 100 per cent. It is a matter of considerable interest, particularly to those engineers directly involved, to have a comparison between the estimated and the ultimate actual cost.

Mr. Harte. Charles Rufus Harte, M. Am. Soc. C. E.—It would be interesting to know the details of the drainage system of those bridges in which the troughs were filled with concrete.

On the Boston elevation of the New York, New Haven and Hartford Railroad, hollow trough bridges, similar to those described in this paper, were used to a considerable extent, and, as in Mr. Wagner's case, there was quite a little difficulty in the matter of drainage.

Each trough had a drainage hole in which a copper thimble was expanded. Not only was it very difficult to keep this thimble tight,

but other and entirely unexpected troubles developed. The thimbles were intended to discharge into a copper trough running parallel with the street, which in turn emptied through leaders into the sewer. The trough was suspended from the bridge by wires attached to the centers of spanners over the top of the trough, and in the first installation these spanners were highest in the center. Condensed moisture gathered on the steel, ran down the wires to the spanner and then, instead of decently dropping into the trough, flowed on top of the spanner and over the edge of the trough and then dropped into the street. It was remarkable how often this occurred, and on what valuable clothing the water managed to fall. Until the cause was located and the trouble corrected by depressing the spanner at the center below the level of the ends, the Claims Department was kept busy with complaints. Mr. Harte.

George S. Webster, M. Am. Soc. C. E. (by letter).—During the past twenty-five years, grade crossings in the City of Philadelphia have been abolished by the co-operation of the City with the railroad companies. Mr. Webster.

An Act of the General Assembly of the State of Pennsylvania, approved June 9th, 1874, authorizes cities and towns of the State to enter into contracts with railroad companies, whereby the lines of railroads may be relocated, reconstructed, or elevated, with the view of abolishing grade crossings and furnishing better railroad service.

Under authority of this general act, city ordinances are passed, authorizing the mayor to enter into such contracts with railroad companies on whose lines it is desired to abolish grade crossings. These ordinances are not general, but apply to the particular line of railroad on which the work is to be undertaken.

The Department of Public Works conducts the negotiations with the railroad companies, prepares the ordinances, in co-operation with the same, has charge of the preparation of plans and contracts for changes of streets and other municipal work, and exercises the right of general supervision and inspection of all construction.

There is no absolute rule which can be laid down as to the best solution in matters of the depression or elevation of railroad lines. It will depend on the physical condition and the character of improvement of the section through which the line passes.

In the abolishment of grade crossings in Philadelphia the method adopted in every case has been one which seemed to suit best the particular problems under consideration.

Practically all the passenger-carrying lines have been elevated or depressed, the exception being branch lines carrying purely local traffic. The lines of the Philadelphia and Reading Railway Company and the Pennsylvania Railroad Company enter their terminal stations over elevated tracks. The Baltimore and Ohio Railroad Company enters its station by tracks depressed below the streets.

Mr. Webster.

Generally, the elevation of the tracks is preferred by the railroad companies, but the depression is somewhat more to the advantage of the City, as the railroad is practically concealed, though the nuisance caused by smoke is greater than with an elevated line.

As a rule, the noise from a depressed line is much less than from an elevated line, and where the streets pass below the railroad there is more temptation for the commission of nuisances than where the streets pass overhead in the open.

The method which seems to affect property most seriously is that which involves a partial elevation of the tracks and a partial depression of the streets, in which cases the adjoining property is damaged to a considerable extent by the change of grade of streets, and a long time usually elapses before these properties recover their value and desirability.

The State of Pennsylvania does not contribute to work of this kind. It has been the general practice for the city and the railway company interested to pay equal proportions of the cost, although this is not an invariable rule. In only one instance has the street railway company contributed.

The work of abolishing grade crossings in Philadelphia commenced about 1887, and since that time three grade crossings of important railway lines have been abolished and about one hundred and twenty street and railway grade crossings have been eliminated.

The first ordinance authorizing the abolishment of grade crossings was approved on March 29th, 1887. This provided for the gradual abolishment of grade crossings and the opening of new streets along the New York Division of the Pennsylvania Railroad, in the northeastern section of the city. The Railroad Company elevated its tracks approximately 9 ft., and the grades of the streets were depressed a like amount, so as to permit of the construction of undergrade bridges. The Railroad Company paid the cost of changing the elevation of the tracks, the construction of the bridges, and all work incident thereto within its right of way. The City graded the street approaches to the bridges and paid the damages to property due to the change of street grades. The work has extended over a considerable period of years, and is not yet entirely completed, therefore the ultimate cost to the City or the railway companies has not been determined, but when the agreement was entered into it was estimated that the cost of the work would be approximately equally divided.

An ordinance of December 26th, 1890, provided for the construction of the Reading Terminal Railroad to Market Street, by an elevated line from Green Street and the abolishment of grade crossings at Columbia Avenue and at Broad Street and Lehigh Avenue, those streets being carried above the railroad. The Railway Company paid all the cost of these improvements and changes.

Mr. Webster.

An ordinance of February 4th, 1892, provided for the abolishment of grade crossings of the North Pennsylvania Railroad and the Connecting Railway Company, together with four grade crossings of streets, at a cost of $300 000, $200 000 being paid by the City and $100 000 by the Pennsylvania Railroad Company, lessee of the Connecting Railway. The Philadelphia and Reading Railway Company, lessee of the North Pennsylvania Railroad, did not contribute.

In abolishing the grade crossings on Pennsylvania Avenue, under the ordinance of March 17th, 1894,* the railroad was depressed below the streets and carried partly in tunnel. Seventeen grade crossings were abolished, at a cost of $5 487 000, the cost being equally divided between the City and the Railway Company.

Under an ordinance of 1897, thirty grade crossings were abolished on the Philadelphia and Trenton Railroad. In this instance the railroad was elevated in such a manner that street grades were not disturbed. The City contributed $800 101 toward the cost of the work and the Pennsylvania Railroad Company, lessee of the Philadelphia and Trenton Railroad, paid the remainder, amounting to $667 541.60.

An ordinance of 1900 provided for the abolishment of four grade crossings on the line of the Chestnut Hill Branch of the Philadelphia and Reading Railway. In this case the contract price was $216 000. The City appropriated $70 000 and assumed the liability for land damages, which amounted to $82 035.75, making the total cost to the City $152 035.75. The Railway Company paid the remainder, $146 000.

The elevation of the tracks of the Philadelphia, Germantown and Norristown Railroad, authorized by ordinance of October 13th, 1906, which has been described so fully and clearly by Mr. Wagner, involves an expenditure of approximately $7 000 000. The cost of elevating the tracks in order to give the railway company "as good accommodations and as complete and convenient facilities as existed prior to the commencement of the work, for conducting business and operating said railroad," was borne equally by the City and the Railway Company, the Railway Company bearing the expense of all betterments to its line.

The Philadelphia Rapid Transit Company (operating the street passenger railway system of the city) contributed $400 000 to the City as a part of the cost of abolishing grade crossings in this instance.

In the work of abolishing grade crossings described in Mr. Wagner's paper it was necessary in several instances to lower the grade of a number of important streets in order to carry them under the railroad. The properties fronting on these streets were used for both manufacturing and residential purposes, and were of moderately high class; and, as some of the streets were important thoroughfares leading from one populous district to another and carried heavy traffic, the problem

* Described in *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 1, and Vol. XLVIII, p. 470.

Mr. Webster. of adjusting grades was difficult. It was decided that on all main thoroughfares the gradients should not exceed 3%, and that on minor residential streets steeper grades, generally not exceeding 5%, would be permissible, and on this basis the street grades were established.

Where streets were depressed to carry them under the railroad, a total reconstruction of the sewer system was made necessary. This required the laying of new trunk sewers (in many cases with flat gradients) to distant points where outlets could be obtained. To prevent overflows in these depressions during heavy rain storms, the sizes of the sewers were designed with an ample factor of safety.

The writer heartily endorses Mr. Wagner's statement that one of the most important features in water-proofing is to get the water away from the structure as fast as possible. As all viaducts and bridges carrying the railroad over streets are required to have solid floor construction, it is important, not only for the maintenance of the steelwork, but also for the protection of pedestrians passing under the bridge, that care should be taken to water-proof the structure properly.

The City of Philadelphia has recently reached an amicable understanding with the Pennsylvania Railroad Company and the Baltimore and Ohio Railroad Company for the abolishment of grade crossings and the readjustment of their freight lines in the southern section of the city, the cost of which is estimated to be about $20 000 000, which, under the terms of the tentative agreement, will be paid for approximately one-half by the City and one-half by the Railroad Companies. The City's contribution in this case is predicated upon the Railroad Companies agreeing that the railroad, to be constructed around the southern section of the city, to replace the existing lines, shall form an "Open Gateway" for the entrance of other railroads into the city, on the payment of an equitable share of the costs.

A minimum clearance of 14 ft. is required under all bridges carrying railroads above streets, whether or not occupied by street railroad cars, but it has been found that a greater clearance is desirable on wide avenues, even at an increased cost.

There has been little or no difficulty in obtaining the co-operation of the railroad companies in this work. Negotiations sometimes move slowly, on account of legal and financial obstacles to be overcome, but, as a rule, the companies are as anxious to abolish their grade crossings as the City is to have them do so, and the work has been planned so that both railway and highway travel has been maintained during construction. In all cases the total cost, including land damages, has been less than the engineers' estimates prepared prior to the commencement of the work.

Mr. Wagner. SAMUEL TOBIAS WAGNER, M. AM. SOC. C. E. (by letter).—Replying to Mr. Snow's questions, the abolition of these grade crossings had been under discussion between the City and the Railway

Company for a number of years prior to the beginning of the work and was the result of a number of conferences terminating in the passing of the ordinance by the City Councils and the signing of the agreement, authorized by the ordinance, by the officials of the City and the Railway Company. During this time the general plan, as finally constructed, was decided on, and the scheme for the division of cost, as well as the necessary legislation required by the City to carry on its part of the work, was perfected. Mr. Wagner.

The ordinance, while general in its scope, covered some points with considerable detail, such as fixing the limits of the four-track viaduct; the main points of the railroad construction in which the City would join; and some specific details of what were to be considered as betterments for the Railway Company, such as the cost of the construction of two tracks of the steel viaduct between Brown Street and Girard Avenue, where the Railway Company had but two tracks prior to beginning work and where it was desired to have four in the completed work. The majority of the cases in which the division of cost had to be determined, was decided on after the work was completely planned and placed under contract. This was done after the most careful preparation of plans and estimates, based on accurate and checked data, as to the dimensions and character of the structures existing before the work was started. Conferences on each subject were held, and the matters decided on were made of final record by the signatures of the Director of the Department of Public Works of the City and the Chief Engineer of the Railway Company, who were the specially designated officials under the terms of the agreement. In this way, before any payments were made on a contract executed by the Railway Company, the amounts to be paid by each interest were definitely determined. As the ordinance did not provide for any betterments to the City, the division of the estimates on the contracts executed by the City was always into two equal shares.

Where streets were widened, the Railway Company paid for one-half of the cost of the widening. This widening was shown on the general plans of the work, which were made part of the original agreement.

The agreement specially provided that the Railway Company could not recover for damages, caused by changes of the grades of streets, to any of its property held prior to the date of the ordinance, and, therefore, whatever amount was necessary to place the property in usable condition was divided equally between the two interests. This work was always done under contract on properly approved plans.

The question of the design of the temporary trestle was one which required much study in order to determine the most economical construction combined with its maintenance under heavy and rapid travel. One of the requirements of the Operating Department was, that there

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should be, at all times, two tracks entirely unobstructed, on which all trains could be run on schedule time. This meant that a perfect track must be maintained, and the primary thought in the design was that of minimum maintenance. As one of the two tracks carried by the trestle was half supported by the masonry wall, and as a portion of it was over a part of the excavation for the foundations, it was considered wise to endeavor to have a specially secure footing for the remainder of the trestle. The section of the work where the trestle was built was formerly occupied by four tracks, 12-ft. centers, on a right of way 66 ft. in width, with drainage ditches on the sides, and was not on a street surface. The bottom of the foundation of the concrete footing was but 12 in. below the old low-level rail, and therefore not down to the original sub-grade. It is believed that any increased cost in the placing of this concrete was more than offset by the saving in maintenance. In one or two cases where the footings were on a fill, and where settlement had occurred, the maintenance of the track by blocking up the stringers was very expensive on account of the continuous traffic.

Mr. Harte's remarks about the water-proofing of the bridges are interesting as being further proof of the perversity of water in finding its own way and not following one which has been provided by the engineer. As already explained, the use of drainage nipples was found to be bad, and the troughs on later designs were entirely filled with concrete and the mastic placed on top. The grades were either toward both ends of the span, or all to one end, depending on the grade of the track. The only difficulty with this method is that if the sub-drainage, behind the back walls, is not good, the water, in case of a heavy rainfall, will back up, and it is difficult to prevent it from working under the apron-plate and down the face of the abutment. Further, it is often difficult to get sufficient grade on even a short span to carry the water off the bridge as rapidly as is desired, especially after the ballast has become dirty. In recent designs, the detail arrangement of the steel above the top of the floor has been made with special reference to the water-proofing, and the principle has been established that it is not wise to allow the water to run off the ends of the span. It should be caught by inlets placed near the ends and carried into the sewers by wrought-iron piping with suitable clean-outs. Intermediate inlets are also provided on the concrete floor, with grades as steep as possible. In these designs the membrane method of water-proofing was adopted as being preferable to the asphalt mastic. In a number of spans of this character just completed, the wisdom of this method seems to be indicated as the results so far have been very good.

It will be noticed that the majority of the bridges described in the paper were water-proofed with a natural rock asphalt mastic, without any protection from the ballast. In spite of what would be

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now considered indifferent details, these bridges are giving fairly satisfactory service, and to the writer's knowledge, there has not been a single complaint from any one as to leakage. It is true that they are not all tight, but the average seems to be good. When it is remembered that there are more than 350 000 sq. ft. of bridge surface over streets, or more than 8 acres in all, it is believed that this is a fair record, when the usual ability of the public to complain, is considered.

The question of the proper material for use in the water-proofing of solid floor bridges is a most interesting one, not only on account of the cost involved, and the uncertainty of the durability of the materials, but also because there are two widely different materials in very general use for such work, namely, coal-tar pitch and materials of an asphaltic nature, and it behooves the engineer to use the best. The whole question is also complicated by the proper design of the steelwork for the water-proofing, as well as the workmanlike application of the materials, both of which are of the utmost importance. With careful observation during the past four years no cases have been discovered where the asphalt mastic has been cut into by the ballast in such a manner as to injure it in any way, and considering the large areas involved, there have been very few cases of the mastic cracking on the main surfaces. Where cracks have occurred, they have generally been along the webs of the girders or at the ends of the spans, and even these have been surprisingly few. In new work where the **V**-shaped wedge of pure, low melting point asphalt has been used along the girders, there has been no cracking or leaking. In asphalt mastic, the writer believes that the use of a ductile, low melting point asphalt as a flux is desirable, and also that the proportions be such as will produce a mastic which is soft rather than one which is hard.

During the summer of 1913, asphalt mastic, made of Val de Travers natural rock, was uncovered on a brick-filled arch carrying a railroad over a street in Philadelphia. The arch had been built by the City in 1892. The mastic was 1 in. thick and placed on cement mortar over the top of the brick. The dry stone packing for drainage was placed immediately on the mastic. Inspection showed that in many cases the stone had cut down into the mastic, but in four openings made at different parts of the bridge, every case of such cutting showed sufficient mastic remaining to give adequate protection. At the end of 21 years of service, the surfaces of this mastic were bright and showed no signs of disintegration of the asphalt. A small sample removed showed surprising ductility and uniformity of the fractured surface.

Replying to the question asked by Mr. Lewis, comparisons between the original estimates of the total cost of any work and the final cost are always interesting, and of great value to the engineer who has much of this kind of work on which to estimate. In this case, it is possible to give but rough figures at the present time, for though

Mr. Wagner. the work of construction is completed, there are a number of suits for damages which are still pending, and until these have been disposed of, final figures cannot be given. It is quite safe to state, however, that the work will be completed well within the original estimated cost of $7 659 740, as at the present time the total expenditures amount to about $6 800 000, and the total amount of the unsettled claims is far less than the difference.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1276

THE INFILTRATION OF GROUND-WATER INTO SEWERS.*

By John N. Brooks, Jun. Am. Soc. C. E.

With Discussion by Messrs. John H. Gregory, Edward S. Rankin, Kenneth Allen, G. L. Christian, E. Kuichling, E. G. Bradbury, Marshall R. Pugh, and John N. Brooks.

An examination of the Index to the *Transactions* of the American Society of Civil Engineers, published recently, shows that the Society has never published a paper on the infiltration of ground-water into sewers. Some recently collected data on the subject are presented in Table 1, and a glance shows that the information is incomplete in most cases, and that there is a wide variation in the form in which it is presented.

The writer, therefore, has attempted to prepare a comprehensive and compact form for the presentation of data on infiltration, and to suggest rational units for the measurement of its quantity.

The factors governing the total quantity of infiltration into a sewer are:

1.—The diameter and length of the sewer.

2.—The material of which the sewer is constructed.

a.—In vitrified pipe sewers, the type of joint used.

b.—In concrete or brick sewers, the type and quantity of water-proofing used.

3.—The skill and care used in laying the sewer.

4.—The character of the materials traversed by the sewer.

5.—The relative positions of the sewer and the ground-water level.

* Presented at the meetings of February 5th and 19th, 1913.

TABLE 1.—Data on Infiltration of Ground-Water into Sewers.

Item No.	Place.	Diameter of sewer, in inches.	Length of sewer, in miles.	Leakage, in percentage of capacity of sewer.	Leakage per day per mile of sewer, in gallons.	Leakage per day per capita, in gallons.	Total leakage, in gallons, per day.	Remarks.
1....	Boston, Mass.	8 to 36	137	.....	40 000	50	5 480 000	Before any connection to sewer.
2....	Massachusetts State	Various.	700	.....	80 000	100	56 000 000	Densely populated section—800 people per mile of sewer
3....	Kalamazoo, Mich.	.....	.....	20%	.....	.....	.....	Before any connection to sewer.
4....	Norfolk, Va.	.....	.....	60%	.....	100	....	
5....	Schenectady, N. Y.	.....	.....	5%	.....	.....	.....	Wet ground and quicksand.
6....	Canton, Ohio.	.....	11	.....	26 500	*44	73 000	
7....	Taunton, Mass.	24	.....	20%	.....	.....	.....	Passing through swamp.
8....	North Brookfield, Mass.	12	0.3	.....	17 000	71	2 500–5 000	
9 ..	Rogers Park, Ill.	6	1.7	.....	1 240	*2	2 108	Deep sockets and careful ramming to minimize leakage.
10 ..	Brockton, Mass.	.....	16	.....	25 000	31	400 000	
11....	Altoona, Pa.	27	1.2	.....	40 814	*65	47 181	
		30	0.6	.....	86 592	*145	52 352	
		33¼ by 44	0.95	.....	264 000	*442	252 342	Brick and concrete sewer. Leakage reduced after careful watching of contractor.
12....	East Orange, N. J.	.....	29	.....	22 400	*43	650 000	10 ft. or more under water. Quicksand. Great precaution to prevent leakage.
13....	East Orange, N. J.	.....	25	.....	8 700	*11	217 500	
14....	Joint Trunk Sewer.	.....	150	10%	25 000	25	3 750 000	Glacial drift and quicksand.
15....	Av. of Bloomfield, Orange, Montclair, and Glen Ridge	.....	.....	.....	.....	107	6 430 000	Average for the years 1895 to 1906.

* 600 people per mile of sewer.

Items 1 and 2. Report of State Board of Health. Discharge of Sewage into Boston Harbor. 1900.

" 3 to 12, inclusive. Report of F. P. Stearns, Past-President, Am. Soc. C. E., on the Sewerage of Mystic and Charles Rivers. Jan., 1899.

" 13 and 14. *Engineering Record*, Vol. 62, Oct. 1st, 1910, p. 377.

" 15. Passaic Valley Sewerage Commission, Report of Dec., 1907, p. 13.

" 14. Joint Trunk Sewer. Takes in sewers of part of Elizabeth, Roselle Park, part of Newark, West Orange, Summit, etc.

Reference to Table 1 shows that in every case data on some of these factors are missing. As it is evidently impossible to express all these factors in precise terms, it becomes necessary to agree upon terms for their expression, and the writer suggests the following:

For the material of which the sewer is constructed, in the case of pipe sewers, use the term "Vitrified Pipe." In the case of concrete sewers, state the thickness of the wall, the mixture, and whether crushed stone, gravel, or slag was used in the aggregate. For brick sewers, state the thickness of the wall, the quality of the brick, and the bond.

The type of joint is usually easily described. If the joints are of cement, it should be noted whether they are finished with a bevel or flush with the bell of the pipe.

For the type and quantity of water-proofing, if a fabric is used, the percentage of the total length of sewer treated may be stated, together with the number of plys of fabric and the kind of paint. In cases where a water-proofing compound is mixed with the concrete, a statement of the kind and quantity of compound will be sufficient.

For skill and care used in laying, use such terms as "Little," "Ordinary," and "Unusual."

For the character of the materials traversed, use such terms as "Wet," or "Dry," "Rock," "Gravel," "Sand," or "Clay."

For the relative positions of the sewer and the ground-water level, reference may be made to the daily reports of the engineer or inspector; and the length of wet trench reported may be expressed as the percentage of the total length of the trench, assuming that where wet trench is reported the sewer will be below the ground-water level.

The foregoing descriptive terms will inevitably have different shades of meaning for different engineers, but the writer believes that they are sufficiently definite to present the salient features of a sewer described by them with a degree of accuracy sufficient for an intelligent comparison with other sewers.

Reference to Table 1 shows that there is a wide variation in the units chosen for the measurement of the quantity of infiltration, and that none of them is altogether satisfactory. Expressing the quantity

of infiltration as a percentage of the maximum capacity of the sewer is irrational, because the maximum capacity depends in part on the gradient of the sewer, which has no relation whatever to the quantity of infiltration.

Gallons per day per mile of sewer is a useful unit only when the size of the pipe and the length of each size are stated, and in a collecting system with many laterals such a statement becomes decidedly cumbrous.

Gallons per day per capita of tributary population appears to be the favorite measure of the quantity of infiltration, but this unit is entirely irrational, for there is no conceivable relation between the number of people contributing sewage and the quantity of ground-water percolating into a sewer.

The total quantity of infiltration, in gallons per day, is a statement which is evidently entirely useless in making comparisons between systems unlike in diameter and length of conduit.

In view of the unsatisfactory form of the data, and of the units thus far considered, the writer suggests the presentation of such data in the manner shown in Table 2, and the following units for the measurement of the quantity of infiltration. These units are used by Mr. Nicholas S. Hill, Jr., and are believed by the writer to be both simple and rational.

For vitrified pipe sewers, the unit is gallons per day per foot of joint. Table 3 shows the total length of joints per 100 ft. of sewer for the commercial sizes of pipe, for which the total length of joints for any length of sewer is obtained by a single multiplication. Evidently, in a vitrified pipe sewer free from cracks, infiltration can occur only at the joints. An examination of Table 1 shows that sufficient data for the computation of infiltration, in gallons daily per foot of joint, are presented in only four cases, namely, Items 8 and 9, and in two sewers under Item 11. The data for these cases are repeated in Table 2, with the computed infiltration expressed in gallons per day per foot of joint; this is seen to vary from 0.3 to 5.0.

For concrete and brick sewers, the unit is gallons per day per square yard of interior surface. Table 4 shows the interior surface area, in square yards per 100 ft. of sewer, for circular sewers, from 3 ft. 8 in. to 6 ft. in diameter, and a single multiplication gives the interior surface area for any length of sewer. For sewers of egg-

TABLE 2.—Data on Infiltration of Ground-Water into Sewers.

Item No.	Sewer. Place.	Type of section.	Material.	Diameter.	Length, in feet.	Skill and care in laying.	Type of joints.	Water-proofing; type and percentage of total length.	Wet trench; percentage of total length.	Character of material traversed.	Infiltration, in Gallons per Day. Per foot of joint.	Per square yard of inside surface.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
8	North Brookfield, Mass.			12 in.	1 584						2.0	
9	Rogers Park, Ill.			6 "	8 976	Unusual.	Deep sockets.				0.3	
11	Altoona, Pa.			27 "	6 336						2.6	
	" "			30 "	3 168						5.0	
16		Circular	Reinforced concrete, stone or gravel, 1 : 2 : 4.	3 ft. 8 in.	2 462	Ordinary.		3-ply felt and pitch, 4 per cent.	80	Wet and dry gravel.		
		"	"	4 " 3 "	7 220	"		"	"	"		0.8
		"	"	4 " 4 "	1 904	"		"	"	"		
		"	"	4 " 6 "	11 066	"		"	"	"		
		"	"	4 " 9 "	5 838	"		"	"	"		
		"	"	5 " 0 "	3 574	"		"	"	"		
		"	"	5 " 3 "	2 901	"		"	"	"		
		"	"	6 " 0 "	12 779	"		"	"	"		

shaped or basket-handle cross-section it is necessary to compute the circumference for the special sections involved, but these computations will have been made in the design of the sewer long before the engineer is ready to make infiltration tests.

TABLE 3.—VITRIFIED PIPE.

Nominal diameter, in inches.	Laying length, in feet.	Length of each joint, in feet.	Length of joint per 100 lin. ft. of sewer, in feet.
(1)	(2)	(3)	(4)
4	2	1.05	52.5
6	2	1.57	78.5
8	2	2.09	104.5
10	2	2.62	131.0
12	2	3.14	157.0
15	2	3.93	196.5
18	2	4.71	235.5
21	2	5.50	275.0
24	2	6.28	314.0
27	2½	7.09	283.6
36	2½	9.42	376.8

TABLE 4.—CIRCULAR SEWERS.

Diameter.	Inside area per 100 lin. ft , in square yards.
(1)	(2)
3 ft. 8 in.	127.99
4 " 0 "	139.63
4 " 3 "	148.35
4 " 6 "	157.08
4 " 9 "	165.81
5 " 0 "	174.53
5 " 3 "	183.26
5 " 6 "	191.99
5 " 9 "	200.71
6 " 0 "	209.44

Item 16 of Table 2 gives the results of a test for quantity of infiltration, recently made by the writer, under the direction of Mr. Hill, on a new reinforced concrete trunk sewer. Triangular weirs were set at two points, 47 745 ft. apart, and each weir was provided with an automatic head-recording device, giving a continuous record of change in head, and reading to the nearest $\frac{5}{1000}$ ft. From these records the average head on each weir for a period of one week was computed, by obtaining with the planimeter the area between the curve of change in head and an arbitrary base line,

and dividing by the proper length. From this average head the corresponding average rate of discharge was computed, making allowance for velocity of approach. The average time of flow between the weirs was obtained by the comparison of maxima and minima points on the respective curves showing decided change of head.

The interval or lag between the time of corresponding discharges having been determined as described, the total discharge at the up-stream weir for a period of one week was then subtracted from the total discharge at the down-stream weir during the corresponding period. The difference in discharge thus determined is a very close measure of the quantity of infiltration between the weirs during one week. The conditions for this test were ideal, as no sewage entered the trunk lines between the weirs during the test. The infiltration found was at the rate of 0.8 gal. per day per sq. yd. of interior surface.

A careful inspection of the length of sewer tested revealed no visible cracks, and the work in general appeared to be first-class, but the quantity of infiltration measured appears to be much smaller than may in general be expected in work of this class.

The writer hopes that discussion of this paper may bring out improvements in the form suggested for the presentation of data on the infiltration of ground-water into sewers, as well as additional data on the subject.

He wishes also to acknowledge his indebtedness to Mr. Nicholas S. Hill, Jr., for valuable criticism and suggestion in the preparation of this paper.

DISCUSSION

Mr. Gregory.

JOHN H. GREGORY, M. AM. SOC. C. E.—The author has presented a paper on a subject which is of especial interest to engineers who are called on to design, construct, or maintain, a system of sewers or a sewage disposal works. Leaky sewers are objectionable from many standpoints. It is not the intention of the speaker to mention all their objectionable features, but to call attention to one of the most important, that is, the consequent increased cost of construction, maintenance, and operation. Due to the infiltration of ground-water, not only must the sewers be larger than would otherwise be required, but also the sewage disposal works, if the sewage receives treatment before final disposal.

The author has opened a way by which many data on the quantity of ground-water leakage into sewers can be obtained if each engineer having any data on the subject will contribute the same in the form of a brief discussion of this paper. The author will then have material which can be tabulated in concise form in his closing discussion, and should be of much service. The speaker believes that many engineers have some data on this subject, but hardly enough to justify them in writing a paper thereon, and, therefore, have not given the information to the Engineering Profession.

With reference to the author's headings under which such data should be tabulated, the speaker has one or two suggestions to make. It would be desirable to include a column for the date—the year alone is sufficient. The engineer will then know how recent is the information he is examining. There should also be a statement of just how the sewer was constructed; if of sewer pipe, cement pipe, or reinforced concrete pipe, the lengths of the pipes, whether of 2, 2½, or 3-ft. sections, etc. Similarly, with monolithic, or other construction, data should be given as to both horizontal and cross-joints.

The speaker, however, is especially interested in the unit in which the leakage is to be reported. Perhaps the unit most commonly used is gallons per 24 hours per mile of sewer. This is very convenient, but it does not go quite far enough—it takes into account the length of the sewer, but not the diameter. The author presents a unit which has much to commend it, namely, gallons per 24 hours per square yard of inside surface. The unit is a rational one, but, unfortunately, is not a very convenient one to use.

The speaker suggests: gallons per 24 hours per inch of diameter of sewer per mile of sewer. A consideration of this unit will show that it takes into account not only length but diameter. He has used it to record the leakage of water pipes, which have been tested under his direction, and has found it extremely convenient. It also has the

advantage that, in designing a system of sewers, calculations of ground-water leakage can be made very rapidly. Mr. Gregory.

To show the application of this unit, Table 5 is presented. This has been computed from the author's data in Tables 1 and 2.

TABLE 5.—Infiltration of Ground-Water into Sewers.

Item No.	Place.	Diameter, in inches.	Length, in feet.	Leakage, in gallons per 24 hours per mile of sewer.	Leakage, in gallons per 24 hours per inch of diameter per mile of sewer.
(1)	(2)	(3)	(4)	(5)	(6)
8	North Brookfield, Mass.....	12	1 584	17 000	1 420
9	Rogers Park, Ill.....	6	8 976	1 240	210
11	Altoona, Pa.....	27	6 336	40 814	1 510
11	" "	30	3 168	86 592	2 890
11	" "	38⅝	5 016	264 000	6 830
16	.....	3 ft. 8 to 6 ft. 0	47 744	7 280	123
17	Columbus, Ohio.....	42	1 744	6 340	151

Item No. 17 in Table 5, gives the result of a leakage test on a 3 ft. 6-in. by 3 ft. 6-in. concrete sewer of horse-shoe section, built under the speaker's direction at Columbus, Ohio, in 1906. The construction of the sewer was in charge of Mr. E. A. Kemmler, Resident Engineer, and the test was made under his immediate direction. This is the speaker's humble contribution to the data which he would like to see tabulated in the author's closing discussion.

It will be seen from Table 5 that the infiltration into the sewers listed varies from 123 to 6 830 gal. per inch of diameter per mile of sewer.

To illustrate the convenience of the unit suggested herein, assume that 300 gal. per 24 hours per inch of diameter per mile of sewer is a reasonable quantity (but the speaker does not care to go on record as stating that this is or is not a reasonable quantity). The leakage into a sewer, say, 18 in. in diameter, would then be 5 400 gal. per 24 hours per mile of sewer, a figure very quickly and easily obtained.

Edward S. Rankin, M. Am. Soc. C. E.—Ten years ago it was the speaker's privilege to listen to a paper on this subject by Mr. A. P. Folwell, before the American Society of Municipal Improvements.* In that paper there was a table giving all the data Mr. Folwell was able to collect; it included gaugings on sixteen sewers. Mr. Brooks gives seventeen gaugings, of which six appear in Mr. Folwell's paper, the net total being twenty-seven. Mr. Rankin.

* "Perviousness of Sewers," *Proceedings*, Am. Soc. Mun. Impvts., 1903.

Mr. Ranl in.

Considering only pipe sewers—to which alone this discussion will refer—and omitting those in which the leakage is only expressed in terms of population or percentage of capacity, neither of which units, as pointed out in the paper, has any direct bearing on the question, there remain but nineteen, and even these are so incomplete as to be practically useless in comparing the leakage of the several sewers mentioned.

The speaker is unable to agree with Mr. Brooks that the single unit of gallons per 100 ft. of joint would form an accurate basis for comparison. Although this includes the two factors of length and diameter, it omits the highly important factor of head of ground-water mentioned on the first page of his paper. The speaker would suggest that the true unit of comparison be expressed by the formula, $n\ d^x\ y\ h$, in which n represents the number of joints, d the diameter of the pipe, in inches, and h the head of water above the invert. From the data available, it would seem to be impossible at this time to determine with any degree of accuracy the values of x and y.

Mr. Brooks considers the leakage as directly proportional to the diameter. Alexander Potter, Assoc. M. Am. Soc. C. E., in his report on the Joint Outlet Sewer, states that, from his observation, the leakage varies as the square of the diameter. The speaker is of the opinion that the true value is less than unity, approximating to the square root of the diameter, and that the greater leakage in the larger pipes of a system is due to these pipes being in most cases laid at a lower level, and consequently being under a greater head of ground-water. He is unable to support this opinion by figures, but it is reasonable to suppose that the leakage is not uniform throughout the joint, but is confined largely to the weakest point, the few inches in the extreme bottom, most difficult to make, and practically impossible to inspect. If this theory is correct, the approximate value of the term, d^x, would become:

For an	8-in.	pipe.....	$2\frac{3}{4}$ in.
	12 "	"	$3\frac{1}{2}$ "
	18 "	"	$4\frac{1}{4}$ "
	24 "	"	5 "

In regard to the other factor, h, there is abundant evidence to show its importance. In soil which is at all porous, every rain storm creates or increases the head, and the most casual inspection of the flow in most sanitary sewers will reveal a decided difference before and after a heavy rain.

A gauging of about 4 miles of 8 to 15-in. pipe sewer in Newark, in March, when the ground was saturated with melting snow, showed a leakage of 38 000 gal. per mile, while in May only 13 500 gal. were found.

Mr. Rankin.

The diagram, Fig. 1, shows the total annual rainfall, in inches, and the estimated average daily leakage, in gallons per mile, on the Newark sections of the Joint Outlet Sewer for the past eight years. Automatic records are kept of the daily flow, and the leakage was obtained by deducting from the total a quantity equal to 500 gal. for each house connection. This is close to the actual quantity, as found in the dryest section of the sewer.

It will be noted that the curves follow each other, showing clearly the close relationship between leakage and ground-water. In the speaker's opinion, the co-efficient, y, would approximate unity, so that the formula would become $n\ h\ \sqrt{d}$.

Although this formula is believed to be theoretically correct, and is useful in comparing the leakage in existing sewers, yet, for practical purposes in designing new work, the more simple form, expressed in gallons per mile or per acre, is to be preferred.

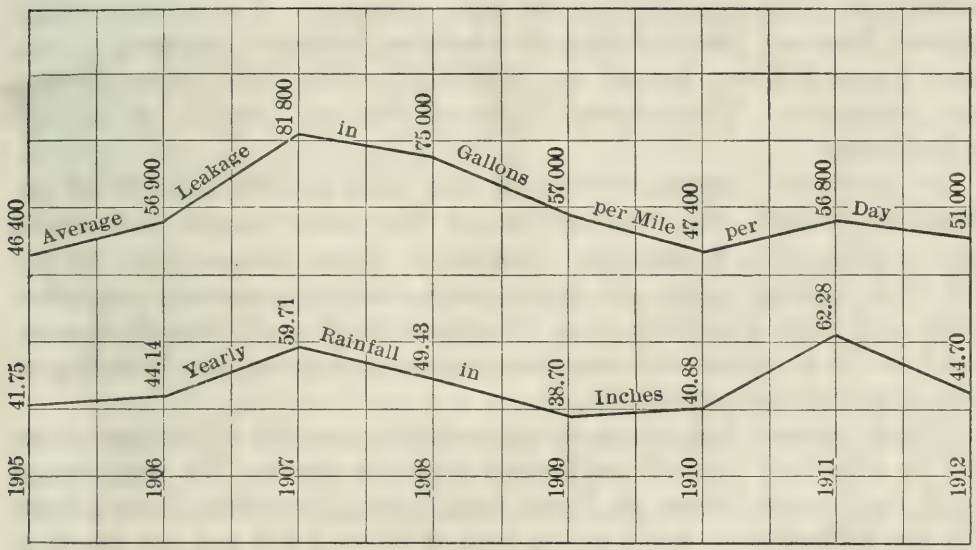

FIG. 1.

Mr. Allen.

KENNETH ALLEN, M. AM. SOC. C. E.—In designing a system of sewers, it is often difficult for the engineer to ascertain what allowance should be made for ground-water. The speaker believes that it is high time that those who have the opportunity to note the conditions affecting the flow due to infiltration do this for the benefit of the Profession.

Data up to the present have been expressed generally in volumes per mile of sewer, or per capita daily, but such all-important elements as head of subsoil water, perviousness of the ground, and the size and character of the sewer itself, are either ignored or stated in such a general way as to be of little value.

Mr. Allen.

There are those who believe that sewers laid in a porous soil, above the level of the ground-water, may be expected to lose an appreciable proportion of their contents by leakage, so that, instead of making an allowance for infiltration, one should more properly make a deduction in the estimated flow due to such leakage. The speaker believes such conditions to be exceptional, and that the ordinary pores left in well-made joints will gradually clog up; thus far, however, there appears to be no consensus of opinion on the subject.

It is desirable to forecast sewage volumes as closely as possible, in order to avoid spending money unnecessarily on sewers which will be too large, on the one hand, and from danger of damage due to backing up of sewage if the sewers are too small, on the other. In any case, excessive infiltration should be prevented, as it not only increases the first cost of the sewers, but, if the sewage is to be pumped or treated, it may increase the annual charges by a very large sum.

In Table 2 the author attempts to standardize the data relative to infiltration. This is a move in the right direction. The speaker would suggest, however, the addition of a column headed "Depth of Invert below Ground-Water Level," an important matter that seems to have been overlooked. The column, "Type of Section," should be omitted, as irrelevant.

No mention is made of one of the most prolific sources of infiltration, namely, the intersection of the sewer barrel with junctions, slants, and bulkheads. Moreover, house connections, if not laid with unusual care, are much more likely to contain imperfect joints, and even fractures, than the sewer itself, and their frequency, therefore, is an important consideration in estimating the quantity of ground-water to be expected.

During storms and thaws an appreciable quantity of surface water may be admitted through perforated manhole covers. In the case of the Joint Trunk Sewer of Essex and Union Counties, New Jersey, this was estimated to enter at the rate of about 1 600 gal. per manhole daily.*

With a system of well-constructed sewers, the initial infiltration is likely to decrease as the pores and crevices in the joints gradually become clogged by silt or deposits of salts, introduced by the ground-water itself or contained in the cement forming the joint. For example, in a pipe sewer recently laid in Batavia, N. Y., with a patent gasket composed of cement and sand placed in pockets of muslin dipped in glue, the infiltration at first was about 1 000 000 gal. per day in 31 miles of sewer, or 32 260 gal. per mile, with the ground-water standing from 3 to 5 ft. above the pipe; but later, after connecting 900 houses with the sewer, this rate fell to about 22 580 gal. per mile.†

* *Engineering News*, February 11th, 1905.

† *Engineering Record*, November 9th, 1912.

Mr. Allen.

The rates of infiltration in Table 6 are abstracted from discussions on the subject by George T. Hammond, M. Am. Soc. C. E., and the speaker,* and may be added to the list given by the author in Table 1.

TABLE 6.

Place.	Rate of Infiltration, in Gallons Per Day.	
	Per mile of sewer.	Per capita.
Alliance, Ohio. 15-in. pipe	195 000	.....
Clinton, Mass.	32 500	62.4
Concord, Mass. 18 % of sewers from 0 to 16 ft. below ground-water level	43 000	.....
Framingham, Mass.	31 180	65
Gardner, Mass.	45 875	156
Madison, Wis. Trunk sewer. Dry weather.	48 000	.....
Malden, Mass. 60% of sewers in wet ground.	50 000	65 to 70
Marlboro, Mass.	59 540	144
Metropolitan (Boston) System, before making connections	40 000	.....
Natick, Mass	77 420 to 100 000	209
New Orleans, La. Good construction. Wet soil.	31 800 to 60 000	.....
Peoria, Ill. Vitrified pipe with brick sewers 15 gal. per sq. ft.	100 000	.....
Providence, R. I.	.....	47
Reading, Pa	5 172	.....
Westboro, Mass. 15-in. sewer in wet ground.	1 320 000	.....
Worcester, Mass. 14 gal. per day per sq. ft. of interior brickwork	31 700 to 800 000	.....

For Baltimore, Md., F. P. Stearns, Past-President, Am. Soc. C. E., recommended from 30 000 to 80 000 gal. per mile per day, but in the actual design of the system it is understood that a per capita rate—usually 20 gal. per day—was used.

It was the opinion of the late Freeman C. Coffin, M. Am. Soc. C. E., that, with 6-in. pipe, and with careful work, the infiltration could be kept down to 4 000 gal. per mile per day.

In the case of a large system of sewerage, it is often more convenient, for general purposes, to assume the daily volume of infiltration per square mile. In the preliminary studies for the sewerage of Baltimore, the speaker used, as suggested by the Consulting Engineers, Messrs. Rudolph Hering and Samuel M. Gray, Members, Am. Soc. C. E., the quantity, 100 000 gal. per sq. mile, increased to 750 000 gal. in the low areas bordering the water front.

More recently, in some studies for the Metropolitan Sewerage Commission of New York, the capacity of sewers has been estimated to provide for 450 000 gal. per sq. mile daily in partly developed districts and 1 000 000 gal. per sq. mile in completely built up territory. Assuming 15 miles of sewer per sq. mile in the former case and 25 in the latter gives: for suburban areas, 30 000 gal., and for urban areas 40 000 gal., per mile of sewer.

* *Proceedings*, Nat. Assoc. of Cement Users, Vol. VII, pp. 690–707.

Mr. Allen.

Assumptions or results in other cities are shown in Table 7.

TABLE 7.

	Infiltration, in gallons per day per square mile.
Louisville, Ky.	1 250 000
New Orleans, La.	1 250 000
Peoria, Ill. (observed)	50 000 to 100 000
San Francisco, Cal. (pipe sewers)	55 300
San Francisco, Cal. (brick sewers)	166 000
Union and Essex Cos., N. J. (observed)	350 000
Washington, D. C. (very wet weather)	620 000
Worcester, Mass.	450 000

The quantity of infiltration, especially for poorly laid brick sewers in a pervious, saturated soil, may be very great. The speaker recalls one case where the water spurted up through the invert of such a sewer in a number of places in jets at least the size of a pencil. On the other hand, concrete sewers may be made practically impervious by the proper proportioning and placing of the ingredients.

Although such sewers will be practically impervious under ordinary heads, it has been thought safer, in some cases, to resort to water-proofing. The first 3 590 ft. of the West Low-Level Interceptor at Baltimore was constructed of concrete with a 4-in. lining of brick. Between the concrete and the brick lining was placed membrane water-proofing consisting of two layers of 10-oz. burlap saturated with asphalt. The sewer varied from 74 to 84 in. in diameter, was provided with numerous house connections aggregating 2 700 ft. in length and was bulkheaded with timber at the upper end. It was laid on a marginal street along the water-front with the invert from 11 to 13 ft. below tide, and included a section of 100 ft. passing just below the bed of a tidal stream. When completed, the infiltration amounted to about 16 000 gal., or 0.223 gal. per sq. ft. of interior surface, per day, probably half of which leaked in around the bulkhead.

With pipe sewers, the only trouble, except with fractures, lies at the joint, and in wet ground such joints should be made with special care. In yielding ground, there should be a special foundation, unless the joint is made of some substance, like bitumen, that will remain to a certain degree plastic. For this purpose, the speaker has used cement, kneaded with tar pitch, with success. About $\frac{1}{4}$ mile of from 8 to 20-in. pipe laid in this way was tested by filling with water and found practically impervious under a head varying from 3 to 4 ft.

In hard ground, a mixture of sulphur and sand, or "leadite," may be preferred.

Whatever material is used should not be subject to deterioration or be either so plastic as to flow from the joint or so rigid as to crack under the conditions in which it is placed.

Mr. Allen.

The entire subject is one involving so many factors, some of them obscure, that probably nothing closer than a fair approximation for considerable lengths of sewer can ever be hoped for; and, for short lengths, a larger margin for error must be allowed. With a knowledge of the local conditions, however, and more complete data regarding existing sewers, the engineer should be able to avoid any serious error of design due to this cause.

Mr. Christian.

G. L. CHRISTIAN, M. AM. SOC. C. E.—The infiltration of ground-water into a sewer depends on so many variable conditions, none of which is exactly similar in any two sewers, that it would seem to be quite impossible to establish a standard of construction from which to determine definitely the degree of impermeability of any two sewers constructed in exactly similar ground, the depth of the ground-water being the same in both cases. It also makes a great difference whether the sewer is a sanitary one, pure and simple, or is part of a combined system. If the former, it is usually desirable to reduce the infiltration to a minimum, the joints being generally tightly caulked with oakum, then filled with cement mortar, and wound with cheese-cloth to hold the mortar in position until it is well set; spurs with loose covers are the exception, rather than the rule. In a combined system, the ends sought are, in a measure, quite different, part of its province being to lower the ground-water level permanently; the joints in pipe sewers are not caulked, being filled with mortar only. For many years it was the custom in the Borough of The Bronx, New York City, to put in a spur on all sewers every 3 ft., except at block intersections. Some years ago this practice was changed, and the distance is now 6 ft. These spurs have loose vitrified covers, and are capable of lowering the ground-water level permanently, regardless of the impermeability of the concrete or brick, or of the joints in pipe sewers.

The design of a combined system of sewers is based, primarily, on the intensity and duration of the maximum rain storm, or on the average of a series of heavy storms, in a given locality, and it is evident, therefore, that the quantity of ground-water infiltration in such a design is so small that it becomes negligible. As to the skill in doing the work, cementing the joints, mixing and placing the concrete, and the character of the aggregate, etc., if one is to depend on the contractor's opinion, it will be first-class in every particular. On the other hand, if the decision is left to the engineers in charge of the work, there are apt to be as many opinions as there are engineers. The character of the trench, as to whether it is wet or dry, is also a relative term, and is apt to be colored by local conditions, that is, what would be termed a wet trench in a region where ground-water is generally high, might mean something quite different if recorded by an engineer doing work in a region where the ground-water seldom appears in the trenches.

Mr. Christian.

A little more than ten years ago, the Sewerage and Water Board of New Orleans, La., made some careful observations, from which it was found that the actual infiltration into the sewers practically checked previous assumptions—that the quantity of ground-water to be provided for would be equal to 0.003 cu. ft. per sec. per acre, which is equal to 1 250 000 gal. per sq. mile in 24 hours. It might be said, in passing, that the ground-water level in New Orleans is very high, and all the sewers are below it. On the foregoing basis, there would be 32 miles of streets in 1 sq. mile, assuming it to be laid out in blocks 400 ft. long and 200 ft. wide, with streets uniformly 60 ft. wide. The infiltration, therefore, would average 39 000 gal. per mile per 24 hours. This figure is also used by the Borough of Brooklyn, New York City; in the Borough of Queens, the assumption is 75 000 gal. per mile per day. Since these observations were made, New Orleans has built some hundreds of miles of pipe sewers, and as all, or substantially all, of its sewage has to be pumped, it is possible to measure quite accurately the quantity of infiltration. It was the speaker's good fortune to witness the laying of some of the pipe, and he noted that the utmost care was taken to make the work water-tight. It was also noted that spurs were put in on each side, every 30 ft., opposite vacant property, the covers being tightly cemented inside the bell. Notwithstanding the care used, there is still considerable infiltration, the actual quantity being shown in the following extract:

GROUND-WATER FLOW, STATION A.*

Year beginning.	Ground-water per mile of sewer, per day, in gallons.
1907	55 000
1908	53 000
1909	51 000
1910	51 000
1911	48 000
1912	42 000

The author describes a concrete sewer in which the infiltration amounted to 0.8 gal. per day per sq. yd. of interior surface, which amounts to 7 300 gal. per mile per 24 hours. From the speaker's experience with the construction of concrete sewers, he would regard this as rather high, unless there was some leakage at the joints, or where each day's work joins that previously laid; and he suggests that a careful inspection would find that to be true.

In conclusion, it may be said that the design of a system of sanitary sewers in any town is a law unto itself; and, in designing such a system, it is wise to consider that the ground-water may have to be taken care of whether one wants to or not. Indeed, in the early stages

*Twenty-fourth Semi-annual Report of the Sewerage and Water Board of New Orleans, La., p. 100.

of such a system, with only a few connections, and many of the sewered streets sparsely populated, it is often desirable to have many unconnected spurs through which water may enter, in order to assist in flushing the sewer. As the district is built up and the houses one by one are connected with these spurs, the need for the ground-water will cease, the house sewage taking its place. Coincident with the building up of the district, it might be assumed that the paving of streets and of court yards is taking place, which would probably mean a corresponding lowering of the ground-water and a reduction of infiltration into the sewer. Mr. Christian.

E. KUICHLING, M. AM. SOC. C. E.—A number of useful suggestions, in regard to the form in which data pertaining to infiltration should be presented, have been made by the preceding speakers, so that little further remains to be said. The principal factors are the character of the soil, the extent to which it is saturated with ground-water, the elevation of the surface of this water above the top of the sewer, and the manner in which the sewer itself is built. An extensive stratum of sandy or gravelly soil often contains and yields a large volume of water, whereas little is delivered from clayey deposits; and in rocky strata seams are frequently encountered which deliver water very copiously. The quantity of water in the subsoil also varies with the daily and seasonal rainfall and the magnitude of the absorptive territory exposed thereto. It is obvious that much more ground-water will be found in the sandy soil at the bottom of a broad valley than in a similar soil on a narrow ridge. Mr. Kuichling.

One of the benefits usually claimed as resulting from a comprehensive system of sewers in a community is the permanent lowering of the existing ground-water table to the level of the sewers, thereby attaining dry cellars automatically and much less dampness of the air in dwellings. Observation has proved that this claim is well founded, and that in most cases shallow wells, in premises adjacent to streets provided with comparatively deep sewers, become permanently dry, even when the utmost care has been taken to make the sewers water-tight. Various reasons for this occurrence have been given in the case of sewers not equipped with under-drains. A common explanation is that the ground-water escapes by flowing along the smooth exterior surface of the main and lateral sewers and finally reaches the outfall; another one is that much less care is given to the construction of the numerous small lateral sewers, whereby the ground-water is enabled to enter through many defective joints and thus find its way into the well-built main sewer; and a third one is that, in the course of a year or two, many small cracks will develop in the entire sewer system, in consequence of settlements and unavoidable changes of temperature, thereby affording the ground-water access to the interior. Each of

Mr. Kuichling.

these explanations is satisfactory in some measure, but it is more likely that all the causes mentioned combine to accomplish the result.

In former years it was a frequent practice to build sewers with open joints at the bottom to admit the ground-water. An interesting measure of the quantity of such infiltration into an extensive sewer system built in this manner is given by Ernest Adam, Engineer of the Department of Streets, of Newark, N. J., in the discussion of a paper on the subject before the American Society of Municipal Improvements, on October 21st, 1903. He stated that many of the old brick sewers of Newark were made without mortar joints in the bottom, the result being a sewage flow of 48 000 000 gal. per day, though the water supply of the city was only 25 000 000 gal. per day. It may be remarked that in 1903 the population of Newark was about 268 500, and that on December 31st, 1903, the total length of all public and private sewers in that city was 198.7 miles.* The total area of Newark was then 14 976 acres, of which 10 679 acres was dry land surface, the remaining 4 297 acres being tidal meadows and water.† From these figures, it will be found that the stated daily infiltration of 23 000 000 gal. of ground-water into the sewer system of Newark corresponds to average rates of 85.7 gal. per capita, 115 752 gal. per mile of sewer, and 2 154 gal. per acre, or 1 378 401 gal. per sq. mile of dry land surface, per day.

It is reasonable to consider that the soil under and adjacent to inhabited buildings will eventually be drained in some manner, either directly by special systems of tiles, or indirectly by defective joints in the lateral sewer pipes; and it is also fair to assume that in most cases the ground-water thus intercepted will be discharged into the public sewers. The total area of the sewered territory, therefore, becomes a factor that must be taken into account, together with the adjacent uplands which may contribute to the flow of ground-water in the populous districts. In some cases much of the flow from the uplands is intercepted by the highest line of sewer, thereby greatly relieving the saturation of the lower ground; but in most instances the sewers are built in streets which cross the contours more or less at right angles, so that ample space is left between such streets for much of the upland ground-water to flow down to the lower districts and seek an entrance into the sewers there.

Some notion of the quantity of flowing ground-water per unit of tributary area may be gained from measurements of the yield of natural springs which have a well-defined water-shed. Numerous observations of this kind in central Switzerland have shown that the normal minimum yield of such springs is 154 gal. per acre of tributary water-shed per day, when derived from the fine-grained sandstone called

* Annual Report, Board of Street and Water Commissioners, Newark, N. J., 1903, p. 118.
† U. S. Census, Report on Cities, 1905, p. 111.

"molasse"; from four to six times that quantity when derived from moraines; and from eight to ten times that quantity when derived from the gravelly soils of valleys. Similar observations in Germany* indicate a minimum yield ranging from 154 to 924 gal. per acre of tributary water-shed per day; an average yield ranging from 462 to 1 232 gal.; and a maximum yield ranging from 770 to 4 620 gal. and upward, depending on the nature of the subsoil and the distance traversed by the percolating water. These yields from flowing springs may also be compared with the observed dry-season flow of rivers. In the New England and Middle States, the least monthly flow of streams having a drainage area of more than 100 sq. miles usually ranges from 0.10 to 0.25 cu. ft. per sq. mile per sec., which is equivalent to a yield of from 101 to 253 gal. per acre per day. This quantity is less than the aforesaid minimum yield of springs, but it must be remembered that in a considerable portion of a large water-shed the soil is generally unfavorable for absorption and percolation. Mr. Kuichling.

A better estimate of the quantity of ground-water that may reach a system of sewers is afforded by the results of numerous land-drainage experiments in England, France, Germany, and the United States. It was found that the quantity of rainfall which would be absorbed by the soil and percolate to the under-drains varied greatly with the character of the soil, the nature of its surface with regard to plant growth, the season of the year, and the magnitude of the rainfall during one or more days. A few references to such data may be of interest. J. Bailey Denton, a well-known English authority, states† that the maximum discharge measured by him from the under-drains of a large tract of free-soil land at Hinxworth took place in January, and was at the average rate of 0.16 cu. ft. per acre per min., which corresponds to 1 710 U. S. gal. per acre per day; and Charnock's observations, of the discharge of field under-drains at Holmfield during 1842-46, exhibit a maximum rate of 0.24 cu. ft. per acre per min., or 2 585 gal. per acre per day. French authorities, like Hervé-Mangon and Delacroix, state that the system of under-drains should have a capacity to remove a depth of at least 0.24 in. of water per day on the entire area, which corresponds to 6 517 gal. per acre per day. In Germany, the Silesian Agricultural Commission recommended that the under-drains should remove not less than 2 845 gal. per acre per day from low-lying flat areas, and 3 390 gal. from higher flat areas. Other German authorities on land drainage, like Vincent and Perels, advise the removal of 6 983 gal. per acre per day.

In the United States the subject of land drainage has recently been investigated by the Department of Agriculture, and in the re-

* Keilhack, "Grundwasser und Quellenkunde," Berlin, 1912, p. 447.

† "Sanitary Engineering," London, 1883 (?), p. 53.

Mr. Kuichling.

port of such observations in Illinois and Iowa, published in 1908,* it is stated that the tile under-drains of areas ranging from 400 to 1 040 acres of black open loam with a clay subsoil frequently gave a maximum discharge of 0.16 in. of depth in 24 hours, or 4 345 gal. per acre per day; also, that in some cases the under-drains were engorged, and that it would be better to remove a depth of 0.25 in. of water in 24 hours, or 6 788 gal. per acre per day. It may be noted that the lines of lateral drains were here from 100 to 250 ft. apart, and it was recommended that in heavier or more clayey soils they should be placed from 40 to 80 ft. apart.

The foregoing discharges from the under-drains of agricultural land were all carefully measured, and can be regarded as a good indication of the quantity of ground-water to be expected in the outskirts of a city. The figures range from 1 700 to 7 000 gal. per acre per day, or from 1 088 000 to 4 480 000 gal. per sq. mile per day, and relate to surface strata of ground from 3 to 4 ft. thick. It is very probable that much larger quantities would be found at times in strata 8 ft. or more in thickness, corresponding to sewered territory. Larger figures would also be obtained by using the results of the numerous percolation experiments which have been made with lysimeters of small area. The data submitted, however, suffice to show that much water is present in the subsoil after a period of heavy rainfall, and that it will enter the sewers under considerable head if opportunity is offered, as in the aforesaid case of the Newark sewers.

Two very instructive measurements of the infiltration into a system of earthenware pipe sewers were made at Stamford, Conn., a number of years ago. An aggregate of 13.38 miles of such sewers, ranging from 6 to 18 in. in diameter, had been laid in 1888 under the direction of the late Col. George E. Waring, Jr., in a wet gravelly soil under an area of 746 acres, or 1.16 sq. miles, of that city, and the infiltration was measured in May, 1889, when only 215 house connections had been made. The quantity was found to be 885 gal. per acre per day, or 566 400 gal. per sq. mile per day, or 49 340 gal. per mile of pipe per day, on the average. In September, 1892, when 530 house connections had been made, it was measured again by Col. Waring and John Bogart, M. Am. Soc. C. E., and was found to have increased to 1 689 gal. per acre, or 1 080 960 gal. per sq. mile, or 94 170 gal. per mile of pipe per day.† The reason for this large increase is doubtless that numerous joints had become defective, in the course of three years, and thus facilitated the admission of the ground-water.

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* Cited also in "Engineering for Land Drainage," by Charles G. Elliot, M. Am. Soc. C. E., Chief of Drainage Investigations, U. S. Dept. of Agriculture. Second Edition, New York, 1912.

† *Engineering Record*, 1892, II, p. 247.

Mr. Kuichling.

Numerous other data, not given in Table 1, will be found in two papers* on the subject by Mr. A. P. Folwell and E. D. Rich, M. Am. Soc. C. E. In Mr. Folwell's paper it is stated that in many sewer systems the infiltration is from 50 to 100% of the volume of house wastes; also that in Boston, Mass., Howard A. Carson, M. Am. Soc. C. E., estimated, from extensive measurements of the flow in the main sewerage system in 1889, that the infiltration was at the rate of 45 gal. per head of population per day, and in Malden, Mass., and Providence, R. I., it was, respectively, 65 and 47 gal. per head per day. It is also stated in the same paper that, in a length of about 4 000 ft. of brick sewer in East Orange, N. J., the rate of infiltration was 570 000 gal. per mile per day, while in 21 miles of 8-in. pipe sewer it was only about 9 000 gal. per mile per day; that in Malden, Mass., it was 50 000 gal. per mile per day on the average for 38 miles of sewer, and that, if only the length laid in wet ground were taken into account, it would be 83 000 gal. per mile per day; that in Westboro, Mass., in 1899, the infiltration into 1 950 ft. of 15-in. vitrified pipe sewer was 489 000 gal. per day, or at the rate of 1 320 300 gal. per mile per day, and in another section of the same sewer, 1 060 ft. long, it was at the rate of 610 000 gal. per mile per day; that in New York City the infiltration into a system of 5 miles of sewers, ranging from 8 to 24 in. in diameter, was found to be about 1 300 000 gal. per mile per day; and that, according to a report made in 1900 by X. H. Goodnough, M. Am. Soc. C. E., to the State Board of Health of Massachusetts, about 137 miles of sewers, ranging from 8 to 36 in. in diameter, had recently been constructed in and near Boston, in which the infiltration was at the average rate of 40 000 gal. per mile per day before any buildings had been connected thereto.

In Professor Rich's paper it is stated that in a 20-in. pipe sewer in Canton, Ohio, the infiltration was at the rate of 70 000 gal. per mile per day; and that in a 12-in. pipe sewer in New Bedford, Mass., it was 17 000 gal. per mile per day. These two papers also refer to some of the other data given in Table 1. In describing the sewage works of Fond lu Lac, Wis.,† George S. Pierson, M. Am. Soc. C. E., stated that the soil in which the sewers were laid was a very retentive red clay, and that the infiltration into the new 24-in. vitrified-pipe outlet sewer was carefully measured before the old sewers were connected thereto. This new sewer had been laid below the level of low water in the river, and was a little more than 7 000 ft. long. The infiltration was found to be less than 3.0 cu. ft. per min., and was regarded as very slight. From these figures we obtain in said 24-in. pipe a rate of 24 370 gal. per mile per day.

* *Engineering Record*, November 7th, 1903, p. 564, and October 1st, 1910, p. 377, respectively.

† *Engineering News*, May 22d, 1902, p. 411.

Mr. Kuichling.

In Table 2, Item No. 16, the author gives the average rate of infiltration into a reinforced concrete trunk sewer, 47 744 ft. long and ranging from 3.67 to 6.00 ft. in diameter, as being 0.8 gal. per day per sq. yd. of inside surface. This information is interesting, but it should be supplemented by naming the locality and describing the nature of the subsoil along the route. It would also have been useful if the author had stated explicitly that the aggregate interior surface of that sewer was 82 123 sq. yd., and that the measured infiltration was about 65 700 gal. per day, or at an average rate of 7 266 gal. per mile per day, which he leaves his readers to ascertain. In comparison with the other observations cited herein, this rate of infiltration is extremely low, and indicates either great water-tightness of the concrete, or an unusually dry subsoil at the time of measurement.

Reference was made in the foregoing to the probable increase in the quantity of infiltration in the course of time. This is to be expected, as small settlements of a sewer usually occur in soft ground, whereby cracks are produced. In the case of large pipes, such cracks are often due to the rapid settlement of the back-fill after a thorough saturation with water. The speaker has seen many miles of pipe sewer laid truly to line and grade originally, and has found them after ten or more years showing numerous irregularities of both line and grade, as well as many fractures. The same is also true of large brick and concrete sewers. In structures of this kind in wet ground, few have ever been built that did not exhibit considerable infiltration in the outset, and it is not likely that they became tighter subsequently if the external head of water remained unchanged. Cracks are likewise formed by the unavoidable contraction of the material when the temperature is lowered for a long period. It must be presumed, therefore, that the infiltration into old sewers will be greater than into new ones, other external conditions remaining the same.

The subject is important, notwithstanding that it has been treated very briefly in most textbooks on sewerage. It becomes specially prominent in a separate system when the sewage has to be pumped or treated in disposal works, and much of the initial capacity of the pipes is taken up by infiltering ground-water. It is hoped, therefore, that the author will succeed in eliciting and arranging in convenient form many additional trustworthy data. The proposition to estimate the infiltration by the square yard of interior surface is good, but it will probably be more convenient to express it in terms of per foot of diameter and a length of 100 ft., or even 1 000 ft. In any event, the nature of the subsoil and the height of the ground-water surface above the top of the sewer should be given, together with the date of the observation, and a brief description of the construction, and its age.

Mr. Bradbury.

E. G. Bradbury, M. Am. Soc. C. E. (by letter).—A sewer which does not carry a much larger flow during heavy rains than in dry weather is an exception, and, as the maximum flow must be provided for in the design, it is important that information regarding leakage should state the weather conditions. Probably most of the data on systems in use are based on the difference between the storm flow and that of dry periods. There is danger, however, of obtaining misleading figures from observations of leakage in new sewers, or in sewers without connections, when the ground-water is comparatively low, for they may show nothing more than the quantity of water which can make its way through the soil to the sewer. This should be borne in mind in comparing gaugings of leakage in the same sewer at different dates.

Under average conditions, few sewers are built which fail to permit the entrance of ground-water as fast as it can get to them, except during a heavy rain. This is demonstrated by the fact that water is seldom encountered in making connections to public sewers; it is also shown by the usual great increase of leakage coincident with storm conditions.

Unless the tightness of the work, rather than the imperviousness of the soil, is the controlling factor in the quantity of leakage, the size of the sewer evidently makes no difference. As this is seldom the case, it is not strange that there has been great difficulty in establishing any relation between diameter and leakage.

The writer has become convinced, much against his will, that, in the average location, with the ordinary method of making cement joints in pipe sewers, even with the most thorough supervision and careful inspection, the ground-water level immediately along the line of the sewer is invariably lowered to the level of the pipe, sloping upward from it at a gradient depending on the porosity of the soil, the surface topography, and other conditions. Under these circumstances, the original height of the ground-water enters into the problem of leakage only as it affects the flow toward the sewer.

It is seldom possible to determine without great effort the actual head on a sewer under working conditions. The writer believes that if data were limited to observations in time of heavy storm, when the tightness of the sewer is really put to test, something more remotely suggestive of a relation between size and leakage than has yet been produced, might result, although the surreptitious connection of roof drains, the inflow at manholes, and other possible means for the entrance of surface water, render this one of the most difficult subjects on which to obtain reliable information.

In the consideration of disposal plant design, it is necessary to have in mind both the ordinary infiltration, which is a function of soil

Mr. Bradbury.

and ground-water conditions, and the leakage in time of storm, which is governed more directly by the tightness of the construction. The one will cause a continuous flow, usually of small consequence, and the other an occasional heavy discharge which may overtax the capacity of piping, weirs, etc., and perhaps derange seriously the operation of tanks and filters. If the writer's views are correct, it is of course entirely fruitless to attempt to reduce normal infiltration to terms of diameter of pipe; but it should be possible, as already suggested, to establish some general relation which would be of value in estimating the maximum leakage during storm periods.

Mr. Pugh.

MARSHALL R. PUGH, M. AM. SOC. C. E. (by letter.)—The information contributed as a result of this timely paper is most interesting; and, in the hope that it may give the author material which will be of use in his closing discussion, the writer submits some observations made by him recently at Ocean Grove, N. J. South of Asbury Park is Neptune Township, the eastern part of which, known as Ocean Grove, is owned by the Camp Meeting Association. West of this are West Grove and Bradley Park, the greater part of which are included in what is known as "Sewer District No. 1." About a year after the construction of the sewerage system, the writer was retained to design sewage disposal works. The infiltration was so important a factor that a number of observations were made in the effort to determine it, and though the results are discordant in one or two particulars, it is thought that this is due chiefly to the impossibility of determining accurately the sewage flow as distinguished from the infiltration flow, other than by an estimate based on the houses sewered; there is also the great probability that one portion of the system is badly blocked up by sand, as will be noted later.

The system is divided into two parts, joining at a pumping station, which is being supplanted by a gravity outfall now under construction. The lengths and sizes of the sewers are shown in Table 8.

TABLE 8.

Item.	Length of north section, in feet.	Length of south section, in feet.	Length of entire system, in feet.	Remarks.
12-in. terra cotta sewer	1 525	3 400	4 925	System built, 1909-1911.
10-in. " " "	.....	1 000	1 000	Pipe in 3-ft. lengths,
8-in. " " "	13 325	23 460	36 785	with cement joints.
4-in. laterals	2 300	5 140	7 440	Two laterals or Y's
Total pipe	17 150	33 000	50 150	every 50 ft. Soil wet,
Brick manholes	36	72	108	sandy loam.

It will be noted that the number of linear feet of joint in the total length of the 4, 10, and 12-in. pipes, in both the north and south

sections of the system, corresponds so closely to the number of linear feet of joint in an equal length of 8-in. pipe, that, for all practical purposes, the system may be considered as consisting of the following: Mr. Pugh.

North section	3.25	miles of 8-in. pipe.
South section	6.25	" " " "
Entire system	9.50	" " " "

The sewers are laid in a soil consisting partly of sandy loam and partly of sandy clay, or clayey sand, underlaid by gravel, and at times there may be a head of from 3 to 5 ft. of water on the pipes at the lower end of the system, and from 1 to 2 ft. at the upper end. The pipes are in 3-ft. lengths, with the ordinary cement joint, and have a slant or lateral on each side about every 50 ft. They were probably laid with only very ordinary care, and about 76 of the manholes were not brought quite up to street grade, being from 4 to 12 in. too low, and 9 of them still have temporary wooden covers. Some of the manholes were very leaky, and two of them had powerful jets of water shooting through the sides.

As nearly as could be ascertained, the infiltration at times was approximately 900 000 gal. per day, and the house sewage, with the few connections as yet made to the system, amounts to about 50 000 gal. per day, divided fairly equally between the two sections. A sewage ejector which was placed at the pumping station has been discontinued, and, during construction, a temporary pumping plant has been erected. From the records of this plant the following details have been obtained. The rainfall data are given in Table 9, and show what an all-important factor is the precipitation.

TABLE 9.—Monthly Averages of Precipitation.

Month. 1912.	Total, in inches.	Greatest in 24 hours, in inches.	Total snow fall unmelted, in inches.
April	4.41	1.06	....
May	6.47	3.21	....
June	1.12	0.70	....
July	4.33	2.53	....
August	3.04	1.46	....
September	3.46	1.42	....
October	6.95	3.12	....
November	3.82	2.27	....
December	5.49	1.94	11.2

Table 10 shows the infiltration in the entire system, in a dry time, before any attempt had been made to diminish the ground-water flow into the pipes.

Table 11 shows the results after the tops of 76 manholes had been raised, and the leaky ones had been made tight.

Mr. Pugh.

TABLE 10.—INFILTRATION IN ENTIRE SYSTEM BEFORE REPAIRS, AND IN A DRY PERIOD.

Date. 1912.	Precipitation, in inches.	Hours of pumping.	Gallons pumped per day.	Approximate infiltration, in gallons per day.	Infiltration, in gallons per day per inch diameter of pipe per mile of sewer.	Infiltration, in gallons per day per inch diameter of pipe per mile of sewer.
July 20.....	....	7	112 000	62 000	6 526	816
21.....	1.21	8.30	136 000	86 000	9 052	1 131
22.....	....	17	272 000	222 000	23 368	2 921
23.....	....	14.40	235 000	185 000	19 473	2 434
24.....	....	10.30	168 000	118 000	12 421	1 552
25.....	....	7	112 000	62 000	6 526	816
26.....	....	8	128 000	78 000	8 210	1 028
27.....	....	8.30	136 000	86 000	9 052	1 131
28.....	0.05	8.30	136 000	86 000	9 052	1 131
29.....	0.01	8	128 000	78 000	8 210	1 028
30.....	....	8	128 000	78 000	8 210	1 028
31.....	Trace.	8.15	132 000	82 000	8 632	1 079

At the time the repairs were made, the south section was disconnected, and Table 12 gives the infiltration in the north section alone.

The much higher infiltration per mile in the north section may be due to the fact that a large part of the south section is in territory which is not built up, and there is reason to believe that a great many stoppages are caused by sand which is washed in at the manholes (the tops of 50 being below the grade of the streets), thus blocking off the escape of the ground-water through the sewers.

Then, too, the outlet of the north section is about 2 ft. lower than that of the south section, and is under a greater head of water. A part of the north section, also, runs through filled-in land which was formerly a swamp.

TABLE 11.—INFILTRATION IN ENTIRE SYSTEM AFTER REPAIRS, AND IN A DRY PERIOD.

Date. 1912.	Precipitation, in inches.	Hours of pumping.	Gallons pumped per day.	Approximate infiltration, in gallons per day.	Infiltration, in gallons per day per inch diameter of pipe per mile of sewer.	Infiltration, in gallons per day per inch diameter of pipe per mile of sewer.
Sept. 20..	0.01	4.35	73 300	23 300	2 452	306
21..	....	5	80 000	30 000	3 158	395
22..	....	5.20	85 300	35 000	3 684	460
23..	Trace.	5.10	82 650	32 000	3 368	421
24..	0.76	6.50	109 300	59 300	6 242	780
25..	1.42	7.10	114 600	64 600	6 800	850
26..	....	7.30	120 000	70 000	7 368	921
27..	Trace.	6.20	101 300	51 300	5 400	675
Oct. 7	No rain in October prior to this.	5.55	94 700	44 700	4 705	588

Mr. Pugh.

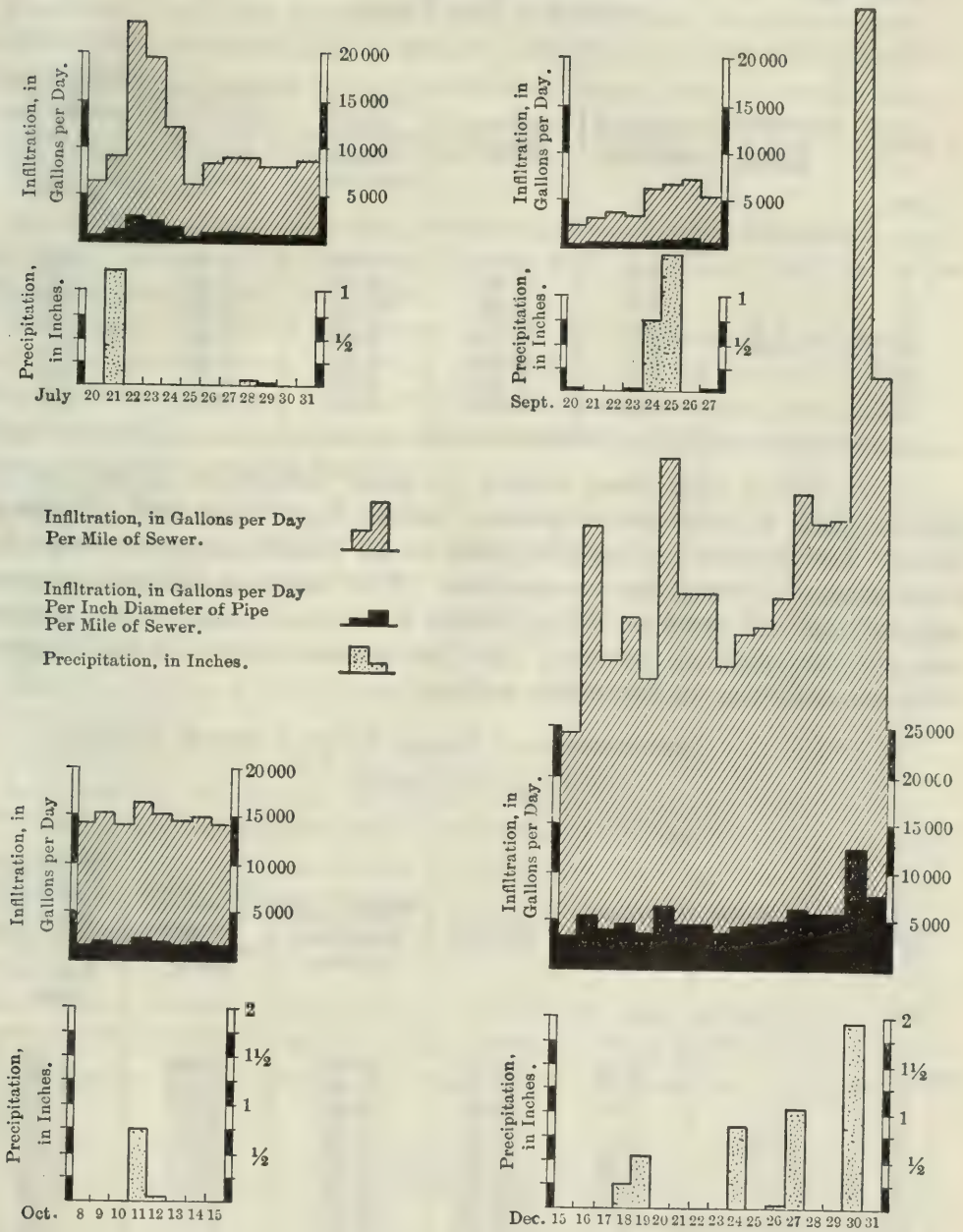
Infiltration, in Gallons per Day.
20 000
15 000
10 000
5 000
Precipitation, in Inches.
1
½
July 20 21 22 23 24 25 26 27 28 29 30 31
Infiltration, in Gallons per Day.
20 000
15 000
10 000
5 000
Precipitation, in Inches.
1
½
Sept. 20 21 22 23 24 25 26 27
Infiltration, in Gallons per Day Per Mile of Sewer.
Infiltration, in Gallons per Day Per Inch Diameter of Pipe Per Mile of Sewer.
Precipitation, in Inches.
Infiltration, in Gallons per Day.
20 000
15 000
10 000
5 000
Precipitation, in Inches.
2
1½
1
½
Oct. 8 9 10 11 12 13 14 15
Infiltration, in Gallons per Day.
25 000
20 000
15 000
10 000
5 000
Precipitation, in Inches.
2
1½
1
½
Dec. 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31

FIG. 2.

Mr. Pugh. It will be noted that the infiltration shown in Table 12 was for a somewhat drier time than that in Table 11.

TABLE 12.—Infiltration in North Section After Repairs, and in a Dry Period.

Date. 1912.	Precipitation, in inches.	Hours of pumping.	Gallons pumped.	Approximate infiltration, in gallons per day.	Infiltration, in gallons per day per mile of sewer.	Infiltration, in gallons per day per inch diameter of pipe per mile of sewer.
Oct. 8.....	....	4.45	72 000	47 000	14 461	1 808
9.....	....	4.40	74 600	49 600	15 261	1 907
10.....	....	4.45	72 000	47 000	14 461	1 808
11.....	0.76	5.0	80 000	55 000	16 930	2 116
12.....	0.01	4.40	74 600	49 600	15 261	1 907
13.....	....	4.30	73 300	48 300	14 861	1 857
14.....	....	4.40	74 600	49 600	15 261	1 907
15.....	....	4.45	72 000	47 000	14 461	1 808

It is obvious that there cannot be more infiltration in the north section than in the entire system; which shows that the estimated sewage flow is open to suspicion, and, in all likelihood, is not properly proportioned between the two sections. The fact that the total pumpage only fell off from 94 700 to 72 000 when two-thirds of the system was cut out, showed, however, the general correctness of the higher infiltration figures in the north section.

TABLE 13.—Infiltration in North Section After Repairs. Winter Conditions.

Date. 1912.	Precipitation, in inches.	Hours of pumping.	Gallons pumped.	Approximate infiltration, in gallons per day.	Infiltration, in gallons per day per mile of sewer.	Infiltration, in gallons per day per inch diameter of pipe per mile of sewer.
Dec. 15...	....	6.30	104 000	79 000	24 308	3 038
" 16...	....	11.	176 000	151 000	46 461	5 808
" 17...	....	8.10	130 600	105 600	32 492	4 061
" 18...	0.26	9.	144 000	119 000	36 615	4 577
" 19...	0.57	7.30	120 000	95 000	29 230	3 654
" 20...	....	12.25	198 600	173 600	53 414	6 676
" 21...	....	9.30	152 000	127 000	39 077	4 760
" 22...	....	9.30	152 000	127 000	39 077	4 760
" 23...	....	8.	128 000	103 000	31 600	3 950
" 24...	0.86	8.40	138 600	113 600	34 900	4 362
" 25...	....	8.45	140 000	115 000	35 300	4 412
" 26...	Trace.	9.25	150 600	125 600	38 600	4 825
" 27...	1.05	11.35	185 300	160 300	49 300	6 162
" 28...	....	11.	176 000	151 000	46 400	5 800
" 29...	....	11.05	177 300	152 300	46 800	5 850
" 30...	1.94	22.	352 000	327 000	100 615	12 577
" 31...	....	16.	256 000	201 000	61 800	7 725

Mr. Pugh.

The latter part of October was wet, and the flows shown in Table 13 clearly demonstrate the effect of the rise in the ground-water in winter.

At first sight it appears strange that the 0.86 in. of precipitation on December 24th did not show in the sewer flow, but it is easily accounted for, as it consisted of a 10-in. fall of snow, on frozen ground, followed on the 27th by a cold rain, which increased the flow somewhat. On the 29th there was a thaw, and a heavy downpour on the 30th, the result being at once visible. The cause of the increase on December 16th is difficult to explain.

Fig. 2 shows graphically the precipitation, and the infiltration, in gallons per day per mile of sewer, and in gallons per day per inch of diameter of pipe per mile of sewer.

These units appeal to the writer as the most convenient and satisfactory for general use, although the latter is open to the serious objection that manholes, slants, and laterals are frequently as numerous on the smaller sewers as on the larger ones, and they undoubtedly are responsible for much leakage.

Mr. Brooks.

JOHN N. BROOKS, JUN. AM. SOC. C. E. (by letter).—In Table 14, which is a summary of the data presented in the discussion of this paper, the writer has included all the sewers mentioned, where sufficient data were given to permit of expressing the infiltration in gallons per mile of sewer. Four sewers in Table 1, for which the infiltration was stated only as a percentage of the total capacity of the sewer, have been omitted.

The writer has examined the original sources of the data presented and has checked the figures in Table 14 with the original statements.

From the very complete data on the sewers of Ocean Grove, N. J., presented by Mr. Pugh, the writer has selected the quantities given in Tables 12 and 13 as representative results, and the figures in Table 14 for the Ocean Grove sewers are averages of the quantities in those tables.

In order to reduce Table 14 to a convenient size, certain abbreviations have been used, as noted at the bottom of that table.

In Columns 15, 16, and 17 the quantity of infiltration is expressed in three units.

In Column 15 it is given in gallons per 24 hours per foot of joint. This is the rational unit for pipe sewers, as suggested in the paper.

In Column 16 it is expressed in gallons per 24 hours per inch of diameter per mile of sewer. This unit is suggested by Mr. Gregory, as an improvement on that proposed by the writer for concrete and brick sewers. The writer accepts Mr. Gregory's unit as rational and convenient for both pipe and concrete or brick sewers. In examining the figures in Column 16 it should be borne in mind that though this

Mr. Brooks.

unit gives a fair comparison between the infiltration in two sewers of similar type, vitrified pipe or concrete, it does not give a fair comparison between sewers of different types, because the entire length of a concrete sewer is porous, and, in a pipe sewer, infiltration is supposed to take place only at the joints.

In Table 5 Mr. Gregory computes the quantity of infiltration expressed in this unit for seven sewers, two of which (Items 11 and 17)

TABLE 14.—DATA ON THE INFILTRATION

Place.	Date of construction.	Date of test.	Type of section.	Material.	Diameter or dimensions, in inches.	Length, in miles.	Skill and care in laying.	Spacing and type of joints.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Boston, Mass.	.....	.....	Cr.	V. P.	8 to 36	137.00	.....	.....
Mass. State	.....	.....	.....	.....	Various.	700.00	.....	.....
Canton, Ohio	.....	.....	.....	.....	.....	11.00	.....	.....
Brockton, Mass.	.....	.....	.....	.....	.....	16.00	.....	.....
E. Orange, N. J.	.....	.....	.....	.....	.....	29.00	.....	.....
E. Orange, N. J.	.....	.....	.....	V. P.	8 to 24	25.00	.....	.....
Joint Trunk Sewer, N. J.	.....	.....	.....	.....	.....	150.00	.....	.....
N. Brookfield, Mass.	.....	.....	.....	.....	12	0.3	.....	.....
Rogers Park, Ill.	.....	.....	.....	.....	6	1.7	Un.	Deep sockets.
Altoona, Pa.	.....	.....	.....	.....	27	1.2	.....	.....
Altoona, Pa.	.....	.....	.....	.....	30	0.6	.....	.....
Alliance, Ohio	.....	.....	Cr.	V. P.	15	.....	.....	.....
Clinton, Mass.	.....	.....	.....	.....	.....	.....	.....	.....
Concord, Mass.	.....	.....	.....	.....	.....	.....	.....	.....
Framingham, Mass.	.....	.....	.....	.....	.....	.....	.....	.....
Gardner, Mass.	.....	.....	.....	.....	.....	.....	.....	.....
Madison, Wis.	.....	.....	.....	.....	.....	.....	.....	.....
Malden, Mass.	.....	.....	Cr.	V. P.	.....	38.00	.....	.....
Marlboro, Mass.	.....	.....	.....	.....	.....	.....	.....	.....
Natick, Mass.	.....	.....	.....	.....	.....	.....	.....	.....
New Orleans, La.	.....	.....	.....	.....	.....	.....	.....	.....
Reading, Pa.	.....	.....	.....	.....	.....	.....	.....	.....
Westboro, Mass.	.....	1899	.....	.....	15	0.37	.....	.....
Stamford, Conn.	1888	1889	Cr.	V. P.	6 to 18	13.38	.....	.....
Stamford, Conn.	1888	1892	Cr.	V. P.	6 to 18	13.38	.....	.....
Canton, Ohio	.....	.....	Cr.	V. P.	20	.....	.....	.....
New Bedford, Mass.	.....	.....	Cr.	V. P.	12	.....	.....	.....
Fond du Lac, Wis.	.....	.....	Cr.	V. P.	24	1.33	.....	.....
E. Orange, N. J.	.....	.....	Cr.	V. P.	10 to 24	.....	.....	.....
Ocean Grove, N. J.	1911	1912	Cr.	V. P.	4 to 12	3.25	Or.	3 ft. cement.
Ocean Grove, N. J.	1911	1912	Cr.	V. P.	4 to 12	3.25	Or.	3 ft. cement.
Newark, N. J.	.....	1903	.....	.....	.....	198.7	.....	.....
E. Orange, N. J.	.....	.....	R.	B.	24 by 36	.....	.....	.....
Westboro, Mass.	.....	.....	.....	B.	.....	.....	.....	.....
Altoona, Pa.	.....	.....	R.	B. & C.	33¼ by 44	0.95	.....	.....
Peoria, Ill.	.....	.....	.....	B. & V. P.	.....	.....	.....	.....
New York City	.....	.....	.....	.....	8 to 24	5.00	.....	.....
Columbus, Ohio	1906	1906	H. S.	C.	42 by 42	0.33	.....	.....
Bronx Valley, N. Y.	1909	1912	Cr.	C. 1 : 2 : 4	44 to 72	9.04	Or.	None.

The following abbreviations have been used in this table: Cr. = Circular, Un. = Unusual, Or. = Ordinary, G. = Gravel, Q. = Quicksand, S. C. = Sandy clay,

have cross-sections other than circular. In these two cases he uses the average of the dimensions of the cross-section stated. If the resulting infiltration is to be compared with that found in sewers of circular cross-section, a fairer comparison will be made by using the diameter of a circle having a circumference equal to the perimeter of the sewer. Assuming the sewers (Items 11 and 17, Table 5) to have rectangular cross-sections of the dimensions given, and using the diame- Mr. Brooks.

OF GROUND-WATER INTO SEWERS.

Water-proofing, type and percentage of length.	Size and spacing of laterals.	Wet trench, percentage of total length.	Average head of ground-water, in feet.	Character of sub-grade.	Infiltration, in Gallons per 24 Hours.		
					Per foot of joint.	Per inch of diameter per mile of sewer.	Per mile of sewer.
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
.....	.....	.....	.....	.....	2.6	1 818	40 000
.....	.....	.....	.....	.....	.....	.....	80 000
.....	.....	.....	.....	.....	.....	.....	26 500
.....	.....	.....	.....	.....	.....	.....	25 000
.....	.....	.....	10	Q.	.....	.....	22 400
.....	.....	.....	.....	.....	0.8	540	8 650
.....	.....	.....	.....	G. & Q.	.....	.....	25 000
.....	.....	.....	.....	.....	2.0	1 420	17 000
.....	.....	.....	.....	.....	0.3	207	1 240
.....	.....	.....	.....	.....	2.6	1 510	40 814
.....	.....	.....	.....	.....	5.0	2 890	86 592
.....	.....	.....	.....	.....	18.7	13 000	195 000
.....	.....	.....	.....	.....	.....	.....	32 500
.....	.....	18	8	.....	.....	.....	43 000
.....	.....	.....	.....	.....	.....	.....	31 180
.....	.....	.....	.....	.....	.....	.....	45 875
.....	.....	.....	.....	.....	.....	.....	48 000
.....	.....	60	.....	.....	.....	.....	50 000
.....	.....	.....	.....	.....	.....	.....	59 540
.....	.....	.....	.....	.....	.....	.....	88 710
.....	30 ft. c. to c.	.....	.....	.....	.....	.....	45 900
.....	.....	.....	.....	.....	.....	.....	5 172
.....	.....	100	.....	.....	.....	88 100	1 320 300
.....	.....	.....	.....	.....	6.0	4 110	49 340
.....	.....	.....	.....	.....	11.3	7 850	94 170
.....	.....	.....	.....	.....	5.0	3 500	70 000
.....	.....	.....	.....	.....	2.1	1 417	17 000
.....	.....	100	5	C.	1.5	1 010	24 370
.....	.....	100	.....	.....	4.7	2 540	43 250
.....	2 - 4 in. 50 ft. c. to c.	100	3	S. C.	2.7	1 890	15 126
.....	2 - 4 in. 50 ft. c. to c.	100	4	S. C.	7.9	5 480	43 764
.....	.....	.....	.....	.....	.....	.....	115 752
.....	.....	100	.....	.....	.....	.....	570 000
.....	.....	.....	.....	.....	.....	.....	415 850
.....	.....	.....	.....	.....	.....	5 390	264 000
.....	.....	.....	.....	.....	.....	.....	100 000
.....	.....	.....	.....	.....	.....	81 400	1 300 000
.....	.....	.....	.....	.....	.....	120	6 340
3 ply felt and pitch 4%....	None.	80	.....	G.	.....	123	7 266

H. S. = Horse shoe, R. = Rectangular, B. = Brick, C. = Concrete, V. P. = Vitrified pipe, C. = Clay.

Mr. Brooks.

ter of an equivalent circle in computing the quantity of infiltration, we have, for the Altoona sewer, 5 390 gal. per day per inch of diameter per mile of sewer, and 120 for the Columbus sewer, instead of 6 830 and 151, as computed by Mr. Gregory.

In Column 17 the infiltration is given in gallons per 24 hours per mile of sewer. This unit is retained as the most useful in cases where insufficient data are presented for the use of the preceding units.

Mr. Allen calls attention to the fact that in a collecting system a large proportion of the infiltration will come from laterals. An attempt has been made to recognize this factor by the addition of Column 11 in Table 14, stating the size and spacing of laterals.

In an impervious soil it may happen that a sewer, however carefully built, will be sufficiently porous to admit ground-water as fast as the soil can carry it. That is to say, the infiltration will be limited, not by the imperviousness of the sewer, but by the quantity of ground-water transmitted to it by the surrounding soil. Mr. Bradbury states that this is usually the case. In such a case the infiltration must be estimated as equal to the ground-water flow, data on which are presented by Messrs. Kuichling and Christian.

The data presented by Mr. Rankin, for the Joint Outlet Sewer, in Newark, and by Mr. Pugh, for the sewers of Ocean Grove, N. J., show that the quantity of infiltration follows closely the rainfall, indicating the direct dependence of the quantity of infiltration on the head of ground-water. The greatly increased infiltration into the Ocean Grove sewers in winter with a high ground-water level, as compared with that of the same sewers in summer, illustrates the same point.

The divergence of opinion among engineers as to the quantity of infiltration to be expected is well illustrated by a comparison of the comments of Mr. Christian and Mr. Kuichling on the infiltration reported by the writer for the Bronx Valley Sewer. Mr. Kuichling says:

"In comparison with the other observations cited herein, this rate of infiltration [0.8 gal. per day per sq. yd. of interior surface] is extremely low, and indicates either great water-tightness of the concrete, or an unusually dry subsoil at the time of measurement."

Mr. Christian says:

"From the speaker's experience with the construction of concrete sewers, he would regard this [infiltration] as rather high, unless there was some leakage at the joints, or where each day's work joins that previously laid; and he suggests that a careful inspection would find that to be true."

As noted in the paper, such an inspection was made, and no leakage was observed; and, further, the subsoil could not have been "unusually dry" because the sewer for its entire length lay beside, and, for a considerable portion of its length, directly beneath, a flowing stream. The low infiltration, therefore, is apparently due to tight concrete.

Mr. Brooks.

The data collected in Table 14 indicate that the quantity of infiltration is dependent on so many conditions, and may vary between such wide limits under apparently similar conditions, that the estimate of this flow in a sewer to be constructed, is a matter calling for the exercise of judgment enlightened by experience, rather than an attempt to apply a mathematical formula.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1277

FREMANTLE GRAVING DOCK: STEEL DAM CONSTRUCTION FOR NORTH WALL.*

By Joshua Fielden Ramsbotham, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. A. R. Archer, Herbert E. Bellamy, and Joshua Fielden Ramsbotham.

Toward the end of 1910, the writer was appointed by the Western Australian Government to design and carry out departmentally the Fremantle Graving Dock, the construction of which was in hand under the Engineer-in-Chief of the Public Works Department, Mr. James Thompson. When the writer took charge, an area had been dredged for the dock to a depth of 49 ft. below low-water mark, the ordinary rise of tide being about 3 ft. 6 in. and extraordinary tides rising only 6 ft. The tidal rise and fall are irregular and intermittent, and, to a large extent, depend on the direction of the wind.

The whole policy of future work depended on the core borings, which had been taken previous to the writer's appointment. From these 73 core borings (two typical bores are shown on Fig. 3) it will be noted that the ground consisted generally of a calcareous sandstone containing pockets of sand, with a bed of clay at a lower level.

After careful consideration and examination of the dredged spoil, the writer advised the Engineer-in-Chief to recommend the Government to abandon the site. From the plan, Fig. 1, it will be noted that by doing this, the entrance and fairway to the harbor would be con-

* Presented at the meeting of April 2d, 1913.

DETAIL OF BORES

Bore No.1 Water 5.80
Sand 28.50
34.30

Bore No.2 Water 7.90
Mud & Seaweed 1.88
Sand 23.79
33.33

Bore No.3 Water 21.03
Sand 13.17
34.20

Bore No.4 Water 5.75
Mud & Sand 8.50
Sand 18.50
32.75

Bore No.5 Water 3.35
Shells 1.17
Mud & Sand 5.00
Sand 23.83
33.35

Bore No.6 Water 3.55
Mud, Sand & Seaweed 2.25
Sand 27.50
33.30

Bore No.7 Water 9.60
Sand 24.00
33.60

Bore No.8 0.80 above L.W.M.
Sand 18.00
Shells & Small Stones 3.00
Sand & Sandstone 14.00
35.00 or 34.20 below L.W.M.

Bore No.9 0.28 above L.W.M.
Sand 12.75
Sand & Shells 1.25
Sand & Sandstone 1.00
Sandstone 4.83
Sand 2.67
Sandstone 10.83
33.33 or 33.05 below L.W.M.

Bore No.10 1.17 above L.W.M.
Sand & Mud 19.75
Sand (Coarse) 8.17
Clay (blue) 3.50
Rock very (hard) 1.50
Sandstone (soft) 0.67
do. (hard) 0.25
do. (soft) 0.58
34.84 or 32.67 below L.W.M.

Bore No.11 Water 0.66
Sand 32.00
Clay Gravel & Washstone 1.33
33.99

Bore No.12 Water 0.30
Sand 22.00
Sand & Mud 8.50
Sandstone (decomposed) 3.00
33.80

Bore No.13 0.77 above L.W.M.
Sand 24.00
Sand & Mud 11.83
35.83 or 33.06 below L.W.M.

Bore No.14 Water 13.50
Sand & Mud 22.25
35.75

Bore No. 15 1.00 above L.W.M.
Sand 13.00
Sandstone & Shells 0.83
Sand & Shells 4.50
Clay (red) 5.42
do. (blue) 8.17
do. (black) 2.50
Rubble 0.25
34.67 or 33.67 below L.W.M.

Bore No.16 Water 19.00
Sand & Mud 14.00
do. Shells 1.00
34.00

Bore No.17 Water 21.30
Sand 12.58
33.88

Bore No.18 Water 3.73
Sand 9.75
Rock (hard) 2.00
Sandstone (soft) 0.33
Rock (hard) 11.00
Sandstone (soft) 5.00
Rock (hard) 0.50
Sandstone 0.41
33.72

Bore No.19 Water 8.25
Sand 21.50
Clay (blue) 3.50
33.25

Bore No.20 Water 0.34
Sand 31.50
Rock 3.25
35.09

Bore No.21 0.68 above L.W.M.
Sand 16.91
Sandstone 2.33
Rock (Very hard 4.23
23.48 or 22.86 below L.W.M.

Bore No.22 1.35 above L.W.M.
Sand 9.25
Rock 2.25
11.50 or 10.15 below L.W.M.

Bore No.23 0.82 above L.W.M.
Sand 14.33
Rock 12.16
26.14 or 25.67 below L.W.M.

Bore No.24 1.00 above L.W.M.
Sand 8.00
Sand & Shells 0.50
Rock (hard) 6.00
(decomposed) Sandstone 13.16
27.66 or 26.66 below L.W.M.

Bore No.25 0.34 above L.W.M.
Sand 9.50
Sand & Shells 0.25
Sandstone 8.50
Rock (Very hard) 5.16
(decomposed) Sandstone 12.00
34.41 or 34.07 below L.W.M.

Bore No.26 Water 4.00
Sand & Shells 2.50
Sand 11.50
Shell 1.00
Sandstone (decomposed) 14.75
33.75

Bore No.27 Water 6.17
Sand & Shells 2.83
do & Mud 22.25
Washstones 2.50
33.75

Bore No.28 Water 0.67
Sand 19.50
(decomposed) Sandstone 8.17
Washstone & Gravel 4.41
32.75

Bore No.29 0.82 above L.W.M.
Sand 21.25
Mud (soft) 4.00
Clay 5.75
34.00 or 33.18 below L.W.M.

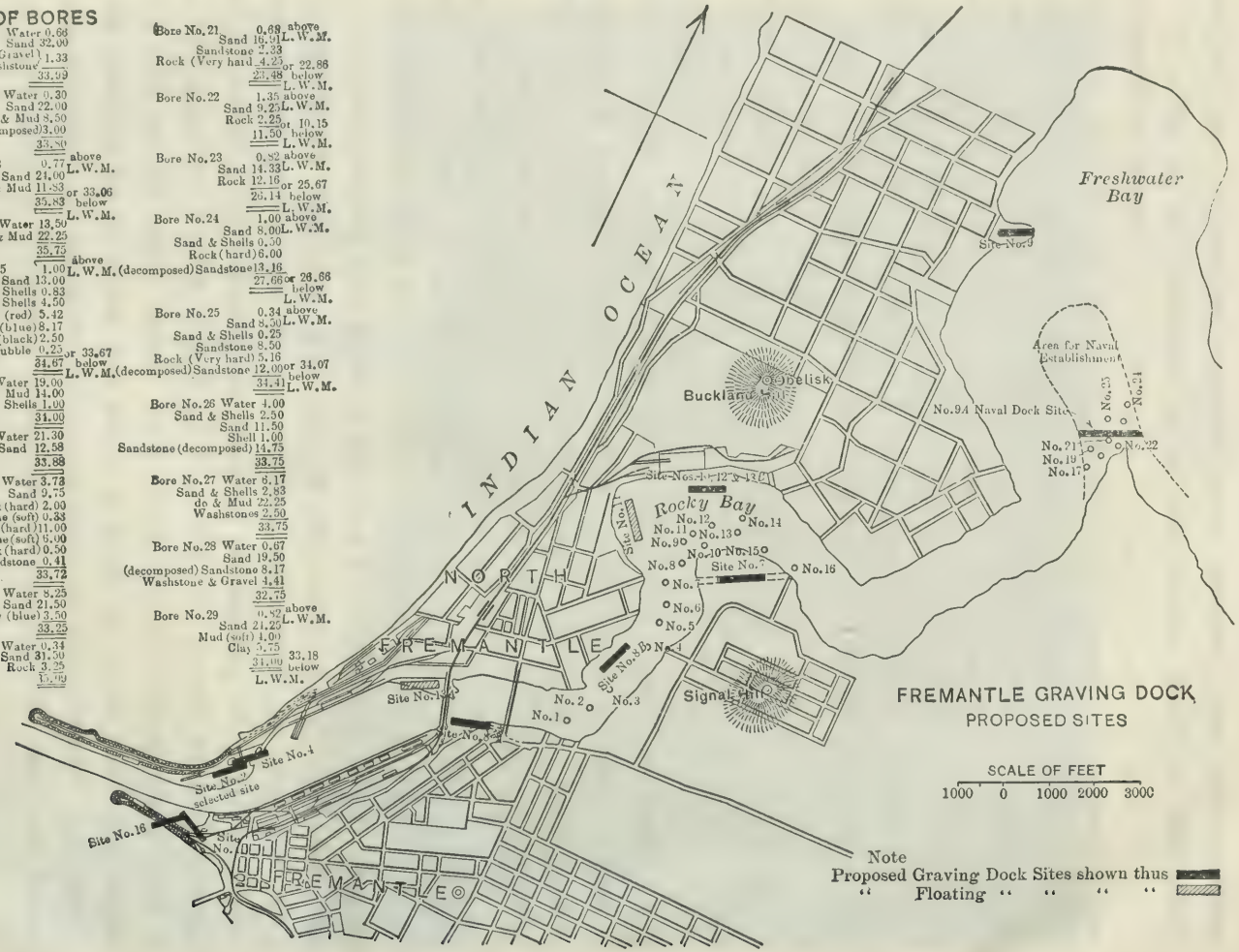

FIG. 1

siderably improved, and the work already done could be turned to advantage.

However, costly enterprises and large undertakings are not to be changed lightly, and it was decided to put down a shaft, make an examination of the bottom, and thoroughly test the porosity of the ground. For this purpose a shaft, 8 ft. 6 in. square, was built, consisting of 12-in. steel piles, 60 ft. long, manufactured by the United States Steel Products Company, the minimum head being 48 ft.

This shaft or dam was unwatered successfully by two 7-in. pulsometer pumps, no puddled clay being used on the outside. On pumping down to 26 ft. below low water, a burst or "blow" occurred under the root of the piles, which, to a large extent, choked itself; but it was sufficiently sealed finally by a "swab" of Italian gasket attached to a sinker, which enabled the shaft to be pumped out. On examination it was found that, at 48 ft. below low water, the rock was hard on the north side, but very soft in the remainder of the area, and it had been noticed that three of the piles had refused to drive home on the north side, while the remainder drove easily, a few so easily (1½ in. at a blow) that, although they were not driven to a set, driving had to be stopped or they would have been under water.

This variation of ground in such a small area was very marked, and, in his report, the writer pointed out that it was problematical whether it would be safe to found a dock in this particular locality, and, further, that there were indications that heavy pumping would be necessary.

On drawing the piles it was noticed that they had penetrated a bed of clay, thus confirming the borings, and also showing that the clay was of good quality.

The writer again advised the Government, on the grounds of the uncertainty of the bottom and of the initial cost, to change the site. It was decided finally by the Government to curtail the sphere of operations to the north wall of the dock, and, in the event of the ground being found unsatisfactory and the ultimate cost prohibitive, to abandon the dock and construct a quay for commerce. At the same time the spit of rock was to be dredged out, thereby considerably improving the harbor.

In work of this character prime cost is very often not sufficiently realized, but in this case estimates were prepared, from data obtained

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FIG. 2.—VIEW OF DAM FOR FREMANTLE GRAVING DOCK.

from similar work, accounting for every item except pumping, which, considering that it was deemed impossible to ascertain, was left as an unknown quantity, with the intimation that the cost would be excessive. From the borings it will be seen that the material in which the piles were driven consisted of a hard limestone cap, coral, clay, and sandstone, the limestone cap being particularly hard in a portion of the pumping station, where a total of 60 ft. of driving had to be done, or, in other words, each pile had to be driven 60 ft. into the ground.

Public tenders were called for, and the lowest, that of the United States Steel Products Company, was accepted.

It is worthy of record that, in driving one of the steel piles, a 45-lb. per yd. steel rail was cut into two pieces, one section being found subsequently on excavating within the dam.

From a driving point of view, these piles are admirable, as the cross-sectional area, or displacement, is small compared with the covering area of the face of the pile, it being very much like driving a knife into a pat of butter, and yet, although whippy, with care very few piles were damaged; and, further, in shipping 1 900 piles, 60 ft. long, from the United States to Western Australia, not one was damaged, and only two were damaged in handling for driving. In handling from the steamer it was found best to have 8 by 3½-in. fish-plates of Oregon pine, 10 ft. long, with a sling chain around them, placed 20 ft. from one end.

In handling, preparatory to driving, a single piece of 7 by 3-in. Oregon pine, 8 ft. long, was found best, and was slung one-third from one end.

Design of Dam.—The design of the dam, to a large extent, was assisted by the results attained in the small trial shaft. The object was to have as few sets of timber as possible, consistent with safety, and as all the shoring had to be done by divers, both from the point of view of economy and speed, this was most important.

The points that require careful watching in designing a dam are:

1.—Strength of skin (which can be taken as a beam supported at several points, and to an extent this is advantageous, as benefit is derived from contraflexure);

2.—Strength of walings or longitudinal beams;

3.—Strength of shores or transverse columns; and,

4.—Factor of safety on the dam.

Great benefit may be derived from working in water, as it is possible to calculate exactly the loads which the various members of the structure have to carry; and, needless to say, the object is to place the walings and shores so that they and also the skin will have an ample factor of safety.

For many reasons, it was decided to use jarrah for the walings and shores, and, as the writer had no previous knowledge of this timber, it was with no small interest that he watched its behavior. As jarrah is a product of Western Australia, a supply could be obtained at the reasonable price of 2 shillings per cu. ft. delivered in railway trucks on the job. It is nearly always heavier than water, and consequently will sink, which is an advantage for divers' work; its strength lies in its outer fibers, therefore it is safer to use hewn than sawn jarrah, as hewers will always hew with the grain.

As heart timber was being accepted in this case, the heart was in the neutral axis or center of the log; further, as jarrah is short in grain, it is certainly advisable not to accept sawn logs, as there is danger of the grain running out unless great care is exercised. In addition to this, as the dam was being built in sections, the divers keeping ahead of the pumped-out sections, the *Teredo navalis* had to be guarded against, and jarrah is the best local timber for resisting this pest.

In designing the dam, a maximum tide of 6 ft. 6 in. above low-water mark was considered (this is seldom if ever reached), and the dredged bottom was taken at 49 ft. 0 in. below low-water mark, thus giving a total calculated head of 55 ft. 6 in., which in actual work could be taken at 51 ft. Further, clay puddle was tipped to a minimum height of 9 ft. at the skin of the dam, in order to make a seal of the porous rock, and, at the same time, reduce considerably the head on the dam. Assuming that the shores are placed at 10-ft. centers horizontally, the pressure on a strip of dam, 10 ft. wide, is:

$$\frac{55 \text{ ft. } 6 \text{ in.} \times 10 \times 55 \text{ ft. } 6 \text{ in.} \times 64}{2 \times 2\,240} = 440 \text{ tons.}$$

The strength of a jarrah beam, loaded uniformly and supported at each end, is given by the following formula, in which K is a constant which has been ascertained to be 13; B is the breadth, and is

assumed to be 14 in.; D is the depth, and is assumed to be 14 in.; and L is the length, which is 9 ft., or 108 in.:

$$\text{Breaking weight, in hundredweights} = \frac{8\,K\,B\,D^2}{L}$$

$$= \frac{8 \times 13 \times 14 \times 14 \times 14}{108}$$

$$= 2\,642 \text{ cwt.}$$

$$= 132 \text{ tons.}$$

Thus, using a factor of safety of 3, the safe load $= \frac{132}{3} = 44$ tons.

Thus the number of walings or beams required is $\frac{440}{44} = 10$.

It is worthy of note that, in calculating the strength of walings, no allowance was made for the fact that the beams were continuous over several supports, thus giving a greater margin of safety than is shown. The shores or columns were 12 in. square and 22 ft. 8 in. long, and were worked out by Gordon's formula in a similar manner, one end being considered rounded and the other fixed, the factor of safety being slightly less than 3.

When the driving of the piles was completed, it was found that it was impracticable to keep them absolutely vertical or in line, due to the hard nature of the ground; and, although the divers packed between all piles and the back of the walings, a certain amount of unquestionable bridging or arching took place. This was so marked that, without any further delay, additional shores were placed alongside those already in, the factor of safety of the walings thus being increased to $3\frac{3}{4}$ and the shores to almost $4\frac{1}{2}$.

The skin of the dam is important, and the largest span was settled empirically from observations taken while pumping out the small trial shaft, and was a clear span of 8 ft. 8 in., subjected to an actual head of 17 ft. 7 in. of water. In the dam in question, the span was 8 ft. 7 in. between the second and third shores. Between the first and second shores the clear span is 9 ft. 11 in., and, although allowance was made for a high tide of 6 ft. 6 in., it is seldom if ever reached. From the writer's experience, a clear span of 8 ft. 8 in., subjected to a mean head of 16 ft. 8 in., is quite satisfactory, the total load on a pile being 4.88 tons, and the maximum stress about 10 tons per sq. in. From the foregoing it may be noted that uniformity has been reached in the temporary dam, and certainly not at the expense of safety.

The embayment for the pumping station offered difficulties outside the ordinary section of the dam, and the writer proposes to touch on it briefly at this stage.

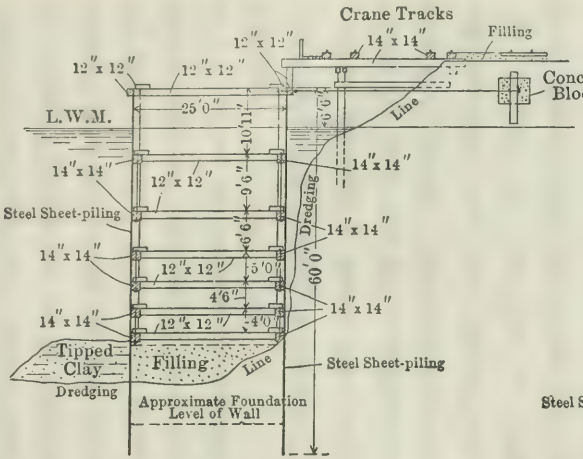
The maximum length of the shores (shown on Fig. 3) is 57 ft. They were built up of 14 by 14-in. jarrah logs, with fish-plates at top and bottom consisting of 20-ft. half timbers of jarrah (14 by 7 in.), without heart, held together with fourteen 1-in. bolts. Care was taken to alter the lengths of the timbers in each line of shores so as to alter the position of the joint; this is important.

In turn, these shores were subdivided, by vertical and horizontal "toms", into small shores 18 ft. in length. Each tier of shores was carefully anchored to the line below by long bolts, and, during excavating, they were invariably kept packed down to the ground by relieving "toms". Further, a deflection of 1½ in. in the center was allowed so as to overcome any tendency of the shore to bend upward or break, and, in addition, there was always considerable weight on the top setting, due to the staging carrying the crane and railway tracks. No trouble was experienced in any way from these shores, the factor of safety on them being 5.27.

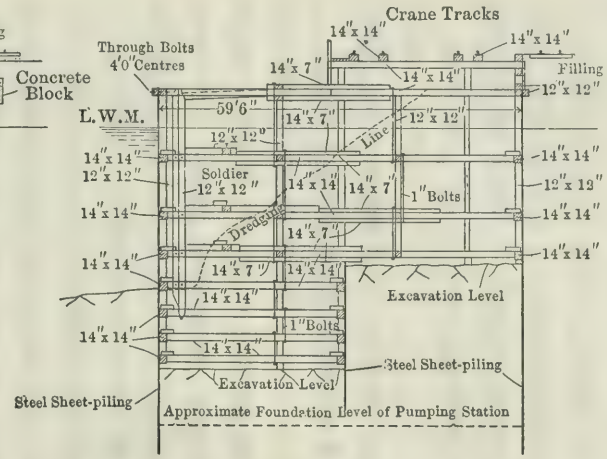
Practical Points on Construction.—Important points were observed during the construction, and perhaps the writer may be pardoned for stating them in some detail.

First, it is of the utmost importance that the female end of the pile should always be driven over the male, and not the male in the female. If the latter is done, a column of spoil will be compressed under the male in the jaws of the female which will finally end in the bursting of the structure. To enable rapid progress two piles were split down the center and a double male pile manufactured, riveting a 6 by 3 by ½-in. T-iron on the back for stiffness. By this means two piling machines were utilized; one was built on an outrigger, with one end supported on staging and the other end on a log on the ground, the piling frame, winch, boiler, etc., being self-contained and easily moved. The other machine was on a barge, and was also self-contained.

In passing, it may be mentioned that the piling winches were made by the Lidgerwood Manufacturing Company (No. 72¼), and were capable of working a 2-ton "tup" or hammer. A 30-cwt. tup was found to be best in practice, and a good man can get 20 blows per



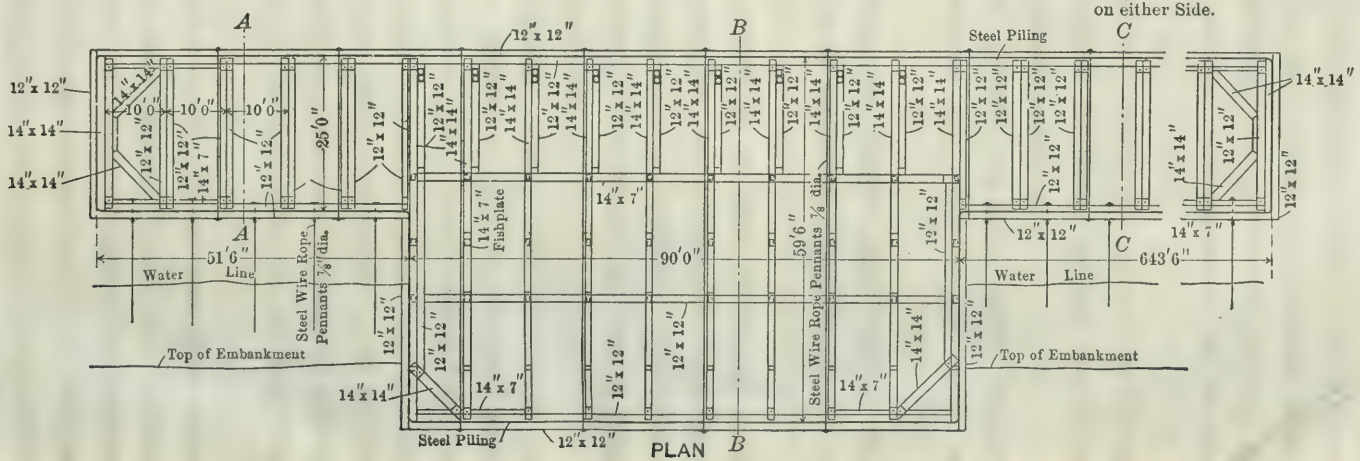
SECTION AT *C-C*



SECTION AT *B-B*

DAM FOR NORTH WALL AND PUMPING STATION FREEMANTLE GRAVING DOCK

Note:- Provision made for Bulkheads every 100 feet to take a Pressure on either Side.



min., a 5-ft. drop being quite sufficient. From the plan it will be noticed that provisions were made for subdividing the dam every 100 ft. by using T-piles, a male and female T-pile being left opposite each other; this was done in accordance with the catalogue of the makers of the piling. When the time came to drive, however, difficulties arose, and as they could be overcome very easily by the manufacturers of the piles, the writer mentions them, hoping that he may thus assist others.

First.—By having a female T-pile, it would mean, in the ordinary course, driving a male pile in it; as previously mentioned, if this is done, it will end in disaster, unless the spoil in which it is driven is a fluid mud. To overcome this, a double female pile was made, and on one side a strip of iron, 1 in. wide and ½ in. thick, was tapped on the outside on one jaw in order to make a lock. The jaws were then interlocked, and the pile was driven, there being clearance for any displacement of spoil between the jaws.

Second.—Great care must be exercised in keeping the two T-piles parallel, otherwise obvious difficulties will arise in closing. The manufacturers could overcome this by making an expanding pile with slotted holes, so as to enable the closing pile to vary its width from the top to the bottom.

Third.—Even with parallel piles, it is extremely hard to marry in with the closing pile, and it is desirable to have a few piles of different width.

Fourth.—If a male T-pile is used at each side of the dam, as should be done, a double female pile is required with which to end, and such should be supplied by the manufacturers.

Fifth.—If a male T-pile is urgently required, it can easily be made, and at small cost, by riveting an ordinary 45-lb. rail to an ordinary pile.

Sixth.—In driving the piles, the moment one seems to be hardening up and has not reached the required depth, another should be driven ahead of it, and, if necessary, this should be repeated until there are signs of having reached soft ground again. It is then possible to exert more force, but even then abuse must not be resorted to, as there is grave danger of parting the piles, but if this does happen by having driven ahead, that danger is limited in extent and is only local.

From the cross-section it will be noted that the piles toward the shore were driven on the side of a cliff, which had been left in a very irregular condition by the dredging, and added considerably to the difficulties of the piling gang; it also rather retarded the progress, as some piles had to be drawn. A fair month's work for the two machines, there being two 8-hour shifts on each machine, averaged 370 piles, representing 9 361 lin. ft. of driving.

The first steel pile was driven on July 14th, 1911, and pumping was started in Dam Section No. 1 on November 16th, the entire dam being framed by the end of that year.

It is interesting to note that 1 872 steel piles were used, representing 47 385 lin. ft., or 9 miles of actual driving. In the first dam section there were 401 piles, which gives 24 060 lin. ft. of joint, representing very nearly 5 miles of joint.

Constructional Work.—At the top of the piles it will be noted that a substantial back-strap is provided, consisting of a 12 by 12-in. jarrah log. This strap was bolted through every fourth steel pile and the inside waling by a 1-in. bolt. Although no trouble was experienced, the writer would recommend 1¼-in. bolts for this purpose.

The walings were hung by two 1-in. bolts from the top timber, breaking joint, and between each there were placed distance pieces or "toms", cut to the required lengths; in this way the necessary accuracy of placing the walings by the divers was assured, and the progress was rapid.

Owing to the fact that the piles were over at the heads and out of plumb, there was a tendency to lift the walings, so that the "toms" and through bolts play an active part, and require most careful watching, as in the event of any small movement, a disaster of great magnitude to life might result, and almost without any warning. No movement was observed, but careful examinations were made and soundings were taken every 10 ft. daily on the clay outside the dam.

When necessary, packings were placed between the walings and the steel piles by the divers, and, when the dam was pumped dry, their work was found to have been done both creditably and satisfactorily.

Across the embayment to the pumping station some interesting work was done. As there was only a sloped bank to which to shore, it was obvious that some other method would have to be adopted. Accordingly, half timbers were bolted on the long shores, top and

bottom, and packed against the top and bottom of the fish-plates which served to build up the long shores and for which purpose fourteen 1-in. bolts were used. This is shown clearly by the section at *B B*, Fig. 3.

King piles, composed of two 14 by 14-in. jarrah logs, bolted side by side, were driven into the ground, and shores were then placed by the diver from the waling to the king pile, and again shored to the bolted half timbers, as previously mentioned, when the dam was pumped down.

As soon as the water was below each setting, the long shores were put in, and all "tomming" and "lacing" done at once.

Pumping Plant.—Three 15-in., centrifugal pumps were used, each capable of working as follows:

Total head, in feet.	Revolutions per minute.	Capacity, in gallons per minute.
40	500	4 900.
50	500	4 400.
60	500	3 800.

The weight of the pumps and engine combined was about 4 tons, so that they were within convenient compass of the 5-ton cranes in use.

Allen "Conqueror" centrifugal pumps were used, and were direct-coupled to "Allen" enclosed two-cylinder compound engines. Each engine developed 110 h.p. at 450 rev. per min. and a steam pressure of 110 lb. per sq. in., the diameter of the cylinders being 10 and 14 in., and the length of the stroke 6½ in.

The boilers were made by Ruston, Proctor and Company, Limited, of Lincoln, England, and at first consisted of two semi-portable, multi-tubular boilers, each having a grate area of 26 sq. ft., capable of evaporating 4 000 lb. of steam per hour, with a pressure of 150 lb. per sq. in., the heating surface being 814 sq. ft. Their length over all was 20 ft. 1 in., their diameter, 4 ft. 6¼ in., and each contained 105 2½-in. tubes. The weight of each boiler without fittings was 9 tons 2 cwt., and with fittings, 11½ tons.

Pumping was started on November 16th, 1911, and continued without intermission until July 23d, 1912. It was found necessary to add another boiler, due partly to the low calorific value of the coal used, but mainly on account of the necessity for cleaning and washing out the boilers, such operations being prejudicial to the continuous running

1905.

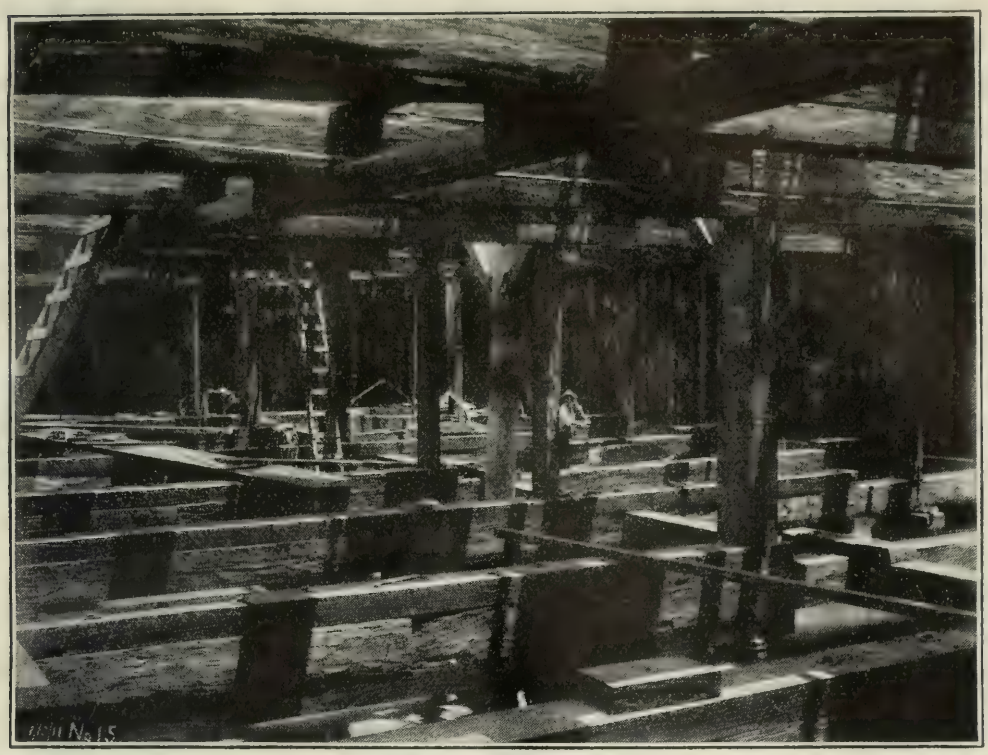


FIG. 4.—EXCAVATION FOR PUMPING STATION, FREMANTLE GRAVING DOCK.

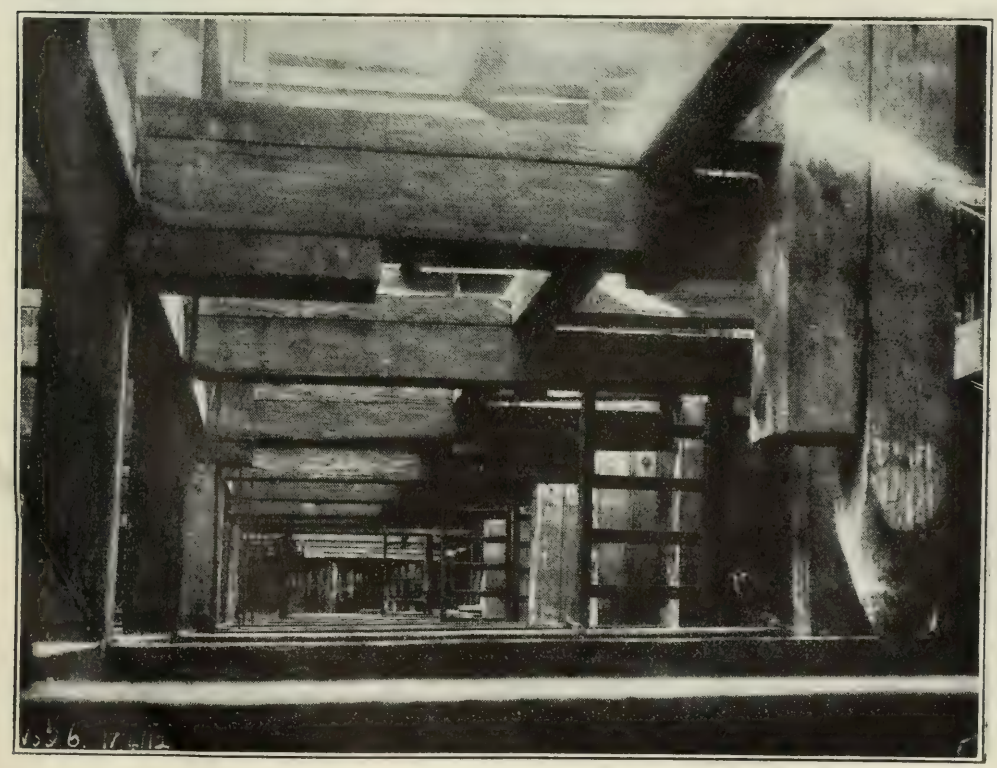


FIG. 5.—METHOD OF SHORING DAM FOR FREMANTLE GRAVING DOCK.

of three pumps. In addition, one 7-in. steam pulsometer pump was used for well sinking, etc.

The Imperial Electric and Engineering Company, of Perth, Western Australia, was the local agent for the above firms, and it is satisfactory to note that from start to finish there were no breakdowns, a result which is most satisfactory when the entire work is at the mercy of the pumps.

Work Done Under Cover of the Temporary Dams.—The pumps were first placed in the bays by the bulkhead, one pump taking up one bay of the dam. They were supported on a staging well braced, checked, and bolted to timber piles driven inside the dam. At first all three pumps were engaged, but gradually a reduction in the pumping took place. The area to be unwatered was 10 453 sq. ft.

After excavating some 20 ft., it was found that, due to the hardness of the rock in that locality, some of the piles had not driven. On reaching a level of 20 ft. below low-water mark, it was decided to excavate a 10 by 7½-ft. well for the temporary pumps, down to 43 ft. below low-water mark, within steel piles, and in the center of the main pumping station, as shown on Fig. 6. This was done, using a 7-in. pulsometer for sinking purposes. Sulphureted hydrogen gas was encountered, which caused very great discomfort to the workmen, some of whom became temporarily blind, others developed eczema, and every one concerned had a bad yellow color and became "as lean as crows", suffering from severe attacks of cramps and general debility. A silver coin placed in the water became of a bronze color in a few seconds. In front of the well, the piles were of steel cut into 5-ft. lengths, and a grating was provided which worked in a groove, so that, when excavating in the body of the dam, a 5-ft. length would be removed, and the pumps were protected by the grating. This answered admirably, and no inconvenience was caused by the silting up of the rose on the pumps.

Unfortunately, after everything had been completed in the shaft, a blow occurred in the bottom of the well, and, although not serious, it was necessary to discover its source. The first thought was that the flow might be coming under the ground where the piles had not driven satisfactorily, or, in other words, the hydraulic gradient had not been cut off. To ascertain this, holes, 30 ft. deep, were drilled 5 ft. away from the skin of the chamber opposite the gap in the piling and 5 centers apart; on drilling to a depth of 28 ft., or 48 ft. below

low-water mark, the drill dropped 2 ft. and water spurted up through the hole. It was thought that the source of the trouble had been found.

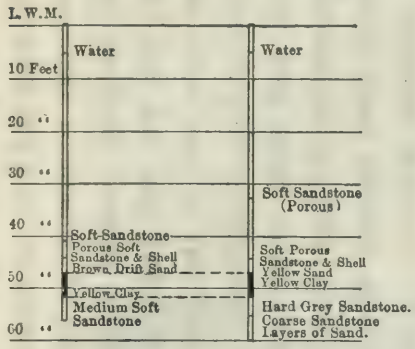
To try and trace the flow of water, a strong solution of potassium permanganate ($KMn\ O_4$) was forced under pressure down the drilled holes, but no results were observed. On mentioning the matter to Mr. E. A. Mann, the Government Analyst, he pointed out that the sulphureted hydrogen would change the purple color of the permanganate rapidly to brown, and then to a colorless solution, this being due to the sulphureted hydrogen ($H_2 S$) changing the permanganate to an oxide of manganese with a deposition of pure sulphur. Some sulphate due to the formation of sulphuric acid ($H_2 S O_4$) is also probably formed.

Mr. Mann kindly gave this question of the presence of sulphureted gas his attention, as it was very noticeable that there was considerable corrosion on the steel piling, in addition to the other features mentioned previously. The writer will refer to this matter later.

For tracing the flow of water, Mr. Mann recommended fluorescein or resorcinolphthalein ($C_2 H_{12} O_5$), a few grains of which gave an extremely bright green fluorescence, which did not lose its color even in a considerable volume of water.

Almost immediately a flow of water was traced in the main dam, a few seconds only elapsing after putting in a few grains of fluorescein before its presence was apparent. Accordingly, systematic drilling and grouting under pressure was adopted, and, as it was novel to the writer, he may be pardoned for referring to the subject in some detail.

Drilling.—It was found advisable, from experience, first to procure a 4-in. iron pipe, from 18 to 24 in. long, and concrete it flush with the ground with a 12-in. cube of concrete, as shown on Fig. 6. This was used for drilling purposes, and prevented the ground from being fretted by the drill; it also stopped any leakage during grouting under pressure. Machine drills and compressed air were available for drilling, but the strata were such as to render them useless, and it was found to be cheaper to jump the holes down by hand. A chisel drill, 3 in. across the bit, was used, and, in the pumping-station area, 30 ft. was the minimum depth of hole drilled. On reaching a level of 48 ft. below low-water mark, as previously mentioned, a cave 2 ft.



TYPICAL BORES

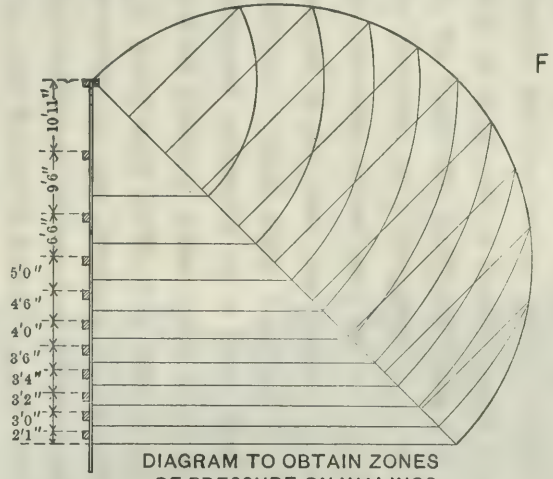
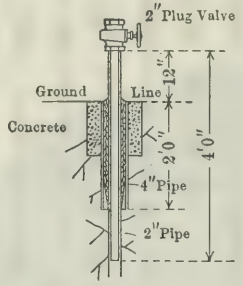
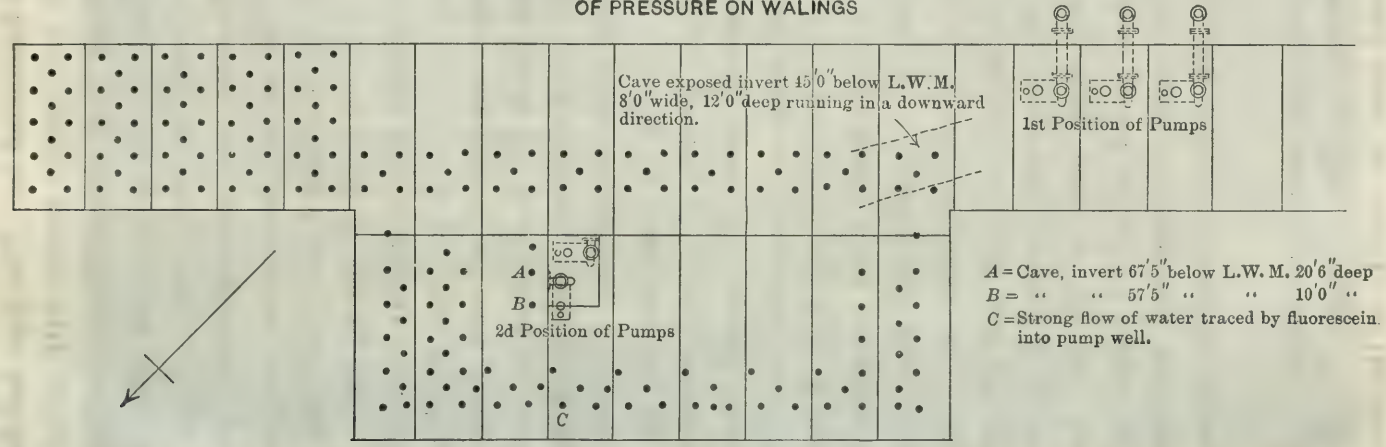


DIAGRAM TO OBTAIN ZONES OF PRESSURE ON WALINGS

DAM FOR NORTH WALL AND PUMPING STATION FREEMANTLE GRAVING DOCK



GROUTING PIPE


Cave exposed invert 15'0" below L.W.M. 8'0" wide, 12'0" deep running in a downward direction.
1st Position of Pumps
A
B
2d Position of Pumps
C
A = Cave, invert 67'5" below L.W.M. 20'6" deep
B = " " 57'5" " " 10'0" "
C = Strong flow of water traced by fluorescein into pump well.

POSITION OF PUMPS, AND PLAN OF GROUTING BORES

FIG. 6.

deep was entered, and a strong flow of water under pressure was set free. The hole having been drilled to the requisite level, an iron pipe, 3 ft. long and 2 in. in diameter, was hammered in. This pipe had been previously roughened on the outside and tightly wound with strips of soft bagging, making a tapered shape very much like an ordinary top; this was well tallowed and tightly bound with strong twine; on the upper end of the pipe was a thread on which was screwed an ordinary 2-in. straight-way valve. In some cases tapered wooden plugs were used until immediately previous to grouting, when they were replaced by the pipe valve. It may be mentioned that there was no trouble from leakage, even under a pressure of 75 lb. per sq. in.

Grouting Machine.—A grouting machine, Fig. 9, similar to the Canniff tank grouting machine* was made. Small modifications, which possibly might be regarded as improvements were introduced, such as having no bends in the pressure pipes, such bends being obviated by having T's fitted with plugs and used for cleaning purposes. When the pipes became blocked, these plugs were found to be invaluable. The Canniff machine consisted of a steel tank, 2 ft. 7¼ in. high and 18 in. in diameter, fitted with a movable hopper on top of a hinged door, a grating 9 in. from the bottom, to prevent lumps from going through the pressure pipes, and an air diffuser again below, which also served the purpose of a grating. The pressure was admitted to a T again by an ordinary coupling, and had a screw-down valve on each side and, in addition, a straight-way valve on the grout exit pipe. By a simple regulation of these valves, air pressures could be admitted either at the top or bottom, by which means the grout was prevented from settling and the clogging of the pipes was reduced to a minimum. Legs were provided on the bottom, and on the top flange rings for slinging, so that the machine could be moved readily by a crane. The tank was built in the Government workshops, and tested at a pressure of 100 lb. per sq. in., the maximum working pressure being 75 lb. per sq. in.

Grouting Under Pressure.—A systematic plan of grouting was arranged (Fig. 6). By using fluorescein, a flow of water had been indicated in a cave subsequently found near the locality of the temporary pumps (first position); in addition to this, a flow of water was

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 404.

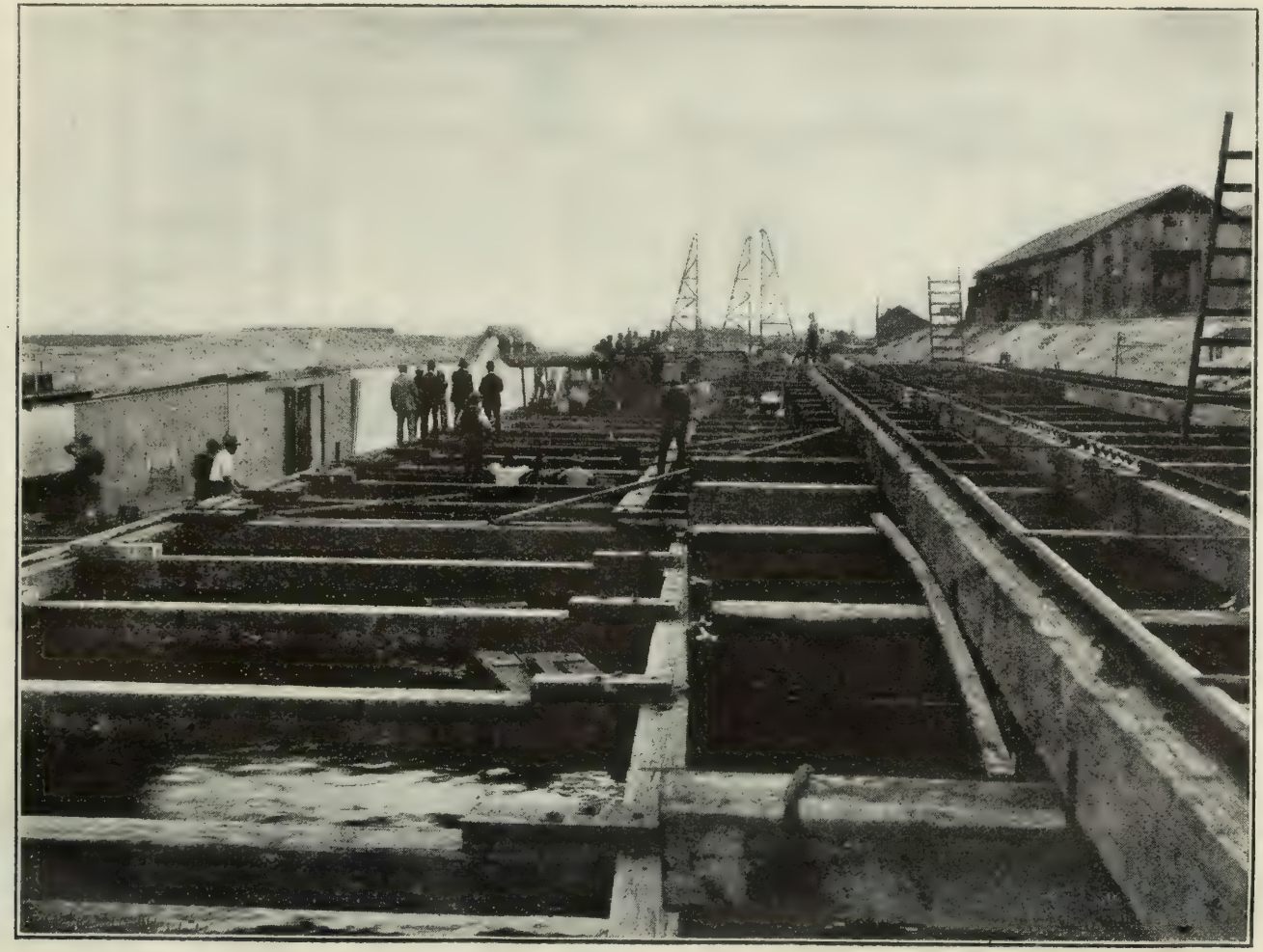
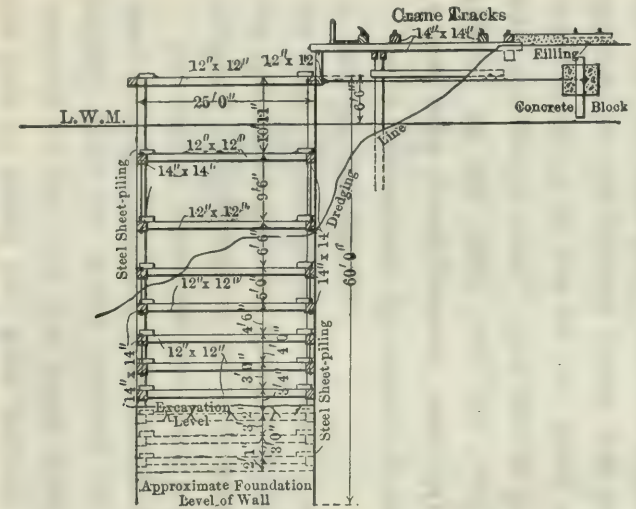
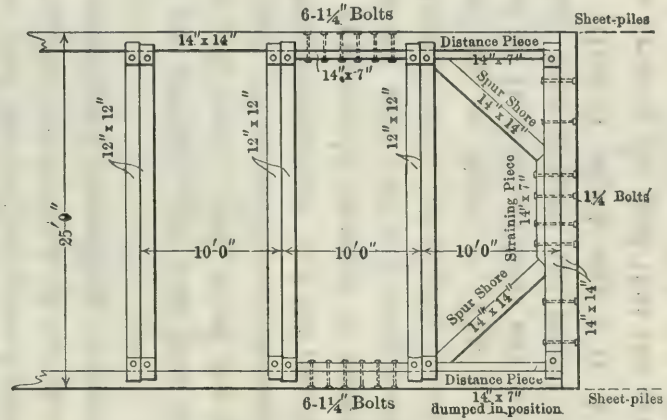


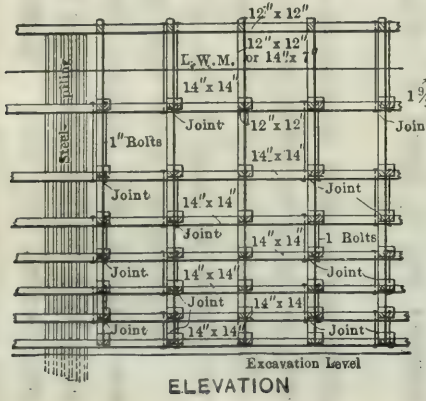
FIG. 7.—DAM AT COMMENCEMENT OF PUMPING.



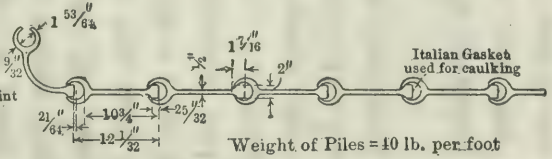
SECTION AT A-A



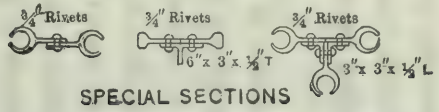
DETAIL OF BULKHEADS AND ENDS OF DAM



ELEVATION



DETAIL OF STEEL SHEET-PILING



SPECIAL SECTIONS

DAM FOR NORTH WALL
AND
PUMPING STATION
FREEMANTLE GRAVING DOCK

FIG. 8.

also found opposite the temporary pump-well in grouting the bore marked *C* on Fig. 6.

On grouting the west bores next to the sheeting, it was found that the flow of water had been entirely stopped. This was proved conclusively by the fact that when grouting holes were drilled 2 ft. 6 in. behind them, no water spouted up, and, further, the drilling was done through layers of hard cement; this was most encouraging.

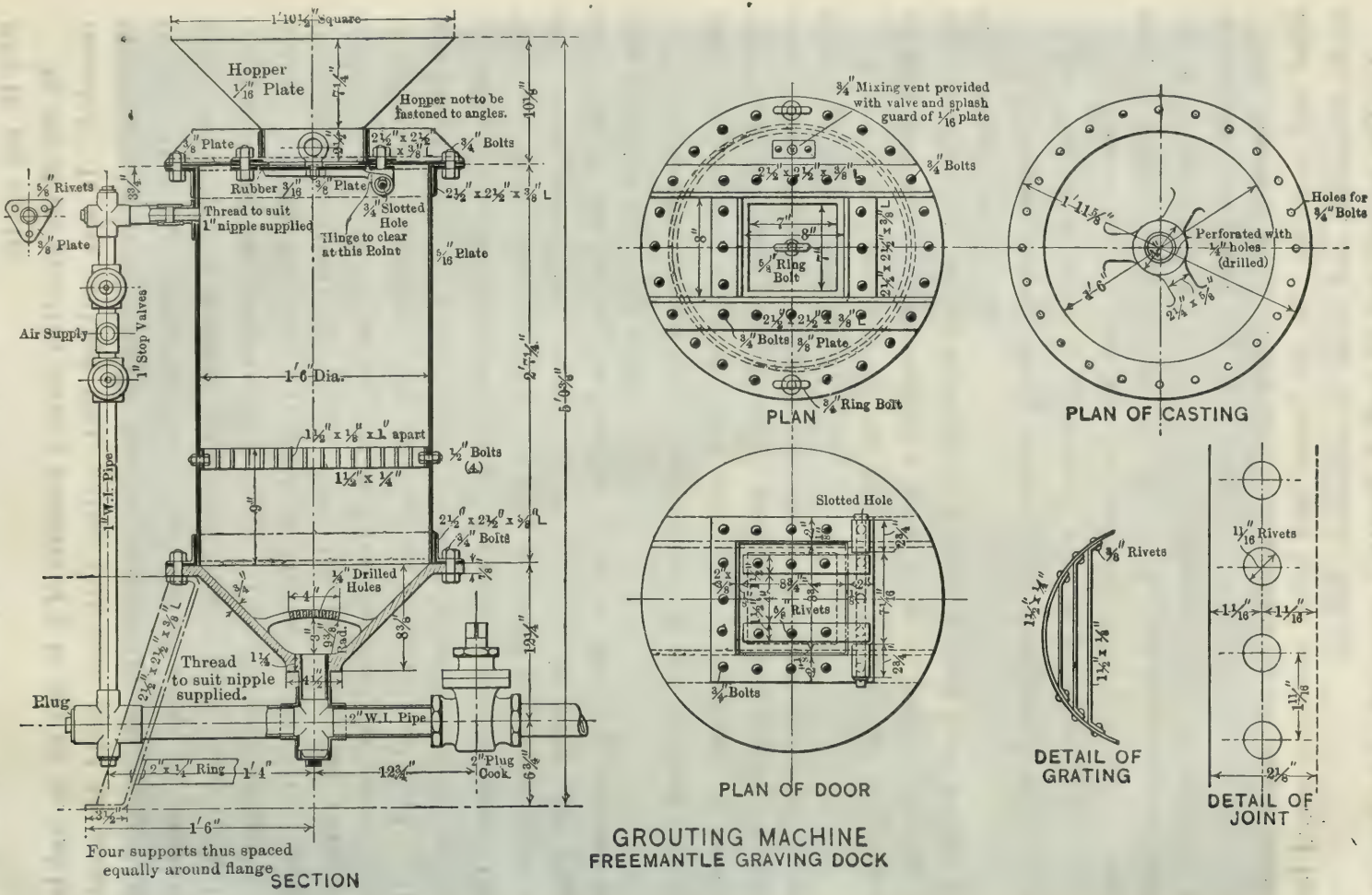
As regards grouting the east side: A strong flow of water was observed, particularly in those grouting holes near the northeast corner, and the entire line was grouted up solid by the machine working from one hole in the center, showing that considerable penetration could be had for the cement.

As a precaution, all valves on the grouting pipes in the vicinity of the pipe being grouted were left open unless they were running freely. In the latter case one valve nearest the pipe being grouted was left partly open. The reason for this was that with a grouting pressure of 75 lb. per sq. in., there would have been grave danger of blowing up the ground which was being grouted, and a valve in the vicinity left open would act as a safety valve. In actual practice it was observed that the surface ground started to crack, due to being in tension, and, but for turning off the pressure promptly, trouble would have been experienced, everything indicating the formation of a miniature volcano.

On an average, each hole took 43 cu. ft. of grout (equal parts of cement and water), and, when the machine ran very freely, saw-dust was added. From subsequent excavations it was shown that the results were most satisfactory, and the penetration of the cement most marked.

An appreciable decrease in pumping was now observable, but the flow in the well was still considerable. It was decided to put down some trial holes in the neighborhood of the burst, and Holes *A* and *B*, were drilled, as shown on Fig. 6. While drilling Hole *A*, the drill dropped 20 ft. 6 in. without any warning, indicating that a fissure or cave had been entered, the invert being 67 ft. 5 in. below low-water mark. Hole *B*, which was 4 ft. 9 in. from Hole *A*, indicated by a sudden drop of the drill that the cave entered was at that place 10 ft. deep, the invert being 57 ft. 5 in. below low-water mark.

A larger hole was then drilled into the cave and a 4-in. pipe was inserted. The water rose in this pipe to within a few feet of the



GROUTING MACHINE
FREEMANTLE GRAVING DOCK

FIG. 9.

level of the water in the river, and the pressure was found to be 9.53 lb. per sq. in.

In one of the grouting bores a pressure of 5 lb. per sq. in. had been recorded. It was realized that, unless this cave could be grouted, the chances of going on with the dock successfully were remote, and, further, that the utmost caution would be necessary in proceeding with the work. A sudden blow might very easily have meant a disaster to all the men working below. Careful preparations were made for filling the cave. A swab of Italian gasket with a sinker attached to marline was forced down the pipe into the cave, and there it was taken by the current and jammed into the exit or burst in the well. Altogether, sixty of these swabs were used, and in this way the burst into the well from the cave was sealed. Then the task of filling the cave was commenced. As previously mentioned, there were two pipes into the cave, on one of which a gravity pipe was fixed above ground level, consisting of a 1-yd. skip fitted with a pipe and valve in the bottom, and on the other the grouting machine was attached with the usual valve.

The fight was started in the early hours of the morning and was continued without stopping until midnight. Altogether, 29 tons of cement were forced into the cave; and, at midday, ashes and water were forced through the gravity machine with a view of saving cement, 6 tons of ashes being used.

A sounding was taken in the well at midnight, and no progress at all was observed. On sending a diver down to inspect the well, he reported that there was practically no leakage, so that some benefit had been attained indirectly.

At this stage several crucial points had been definitely established:

1.—The job would have to be dealt with in small sections.

2.—That being the case, with the pumping plant available, it would take at least 8 years to complete the work, and to overcome this it would be necessary to multiply the pumping plant in order to get several sections under way at a time.

3.—The cave would either have to be grouted up or opened out and filled with concrete. As the former had failed, the latter would have to be attempted, and would be extremely difficult.

4.—The ground was of such a doubtful character as to render it necessary to grout all over the site, which having been done, the safe

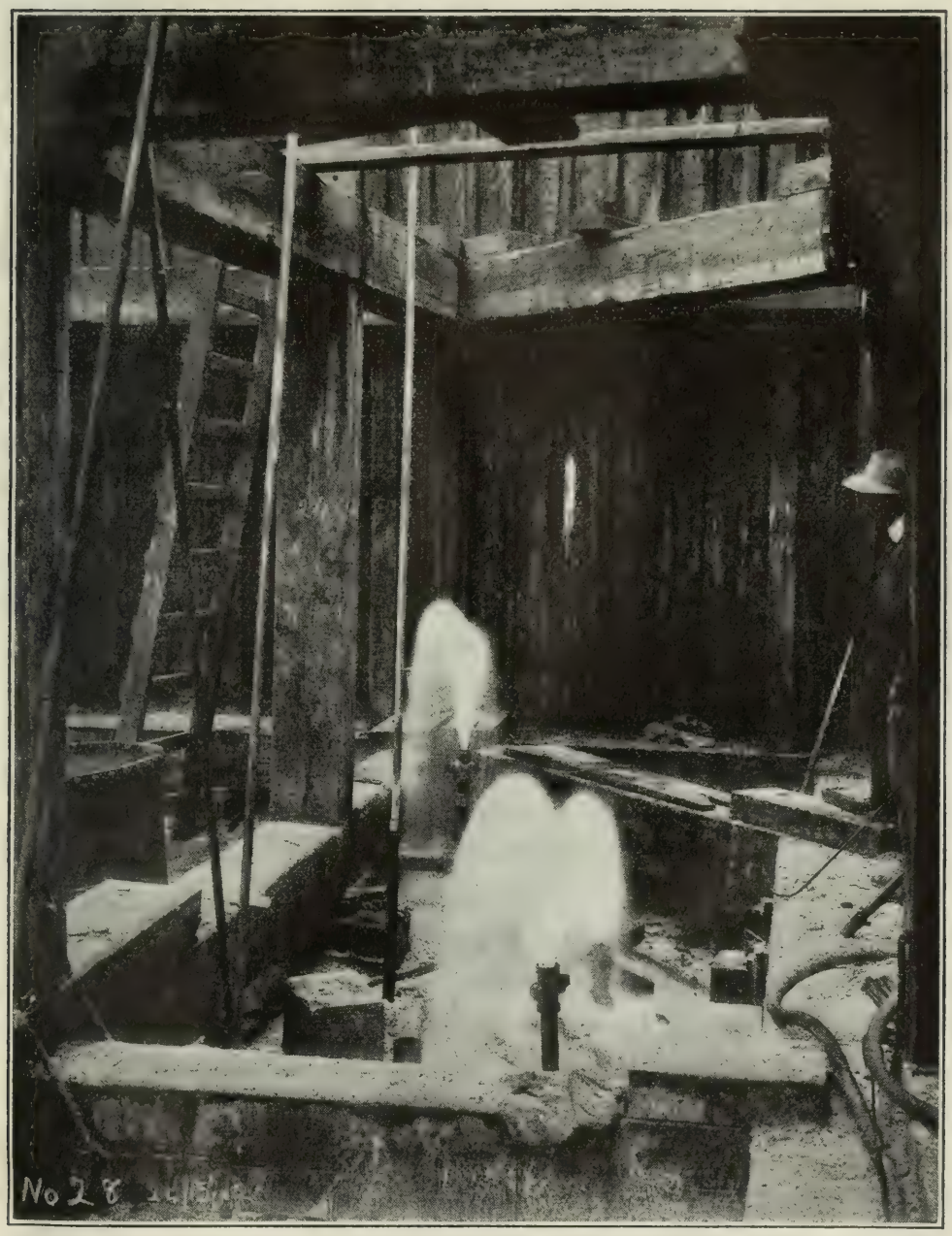


FIG. 10.—DRILLED HOLES SUBSEQUENTLY GROUTED SUCCESSFULLY.

permissible load that it would carry would be low, and the writer was doubtful whether 3 tons per sq. ft. could be exceeded.

5.—Corrosion due to sulphureted hydrogen gas was going on so rapidly as to render the re-using of the front steel piles more than doubtful.

6.—Steps would have to be taken to decrease the pressure of the sulphureted hydrogen gas. This gas caused the writer considerable anxiety, and it is necessary to remember that the stability of the floor depended on the concrete being fortified by steel, and such fortified concrete had to be designed to withstand a hydrostatic pressure from below and a statical load due to a vessel on the keel blocks.

This latter was necessary owing to the ascertained uncertainty of the ground and the presence of caves. Therefore, if this steel in the ferro-concrete was attacked by corrosion accelerated by sulphureted hydrogen gas, the consequences would be serious.

A supplementary estimate was given to the Government, which brought the estimated cost of the dock from its inception up to £725 000, and the writer advised the Engineer-in-Chief to recommend the Government to abandon the site and look elsewhere.

From a harbor point of view, the site was a bad one, and if, as the writer firmly believes will happen, liners in the Australian trade materially increase in size, it is more than probable that either the dock or the Port of Fremantle would have to be moved.

Bearing in mind that a great deal of the work done on the site, such as dredging, would be reproductive, the Government decided:

1.—To look elsewhere for a dock site;

2.—To build a quay on the north side of the dock site capable of berthing any mammoth liner afloat, and as such berth was at the mouth of the river, it would entail the minimum amount of dredging for an entrance to the deep-water berth; and

3.—To dredge away everything to the south and west and so very materially improve the harbor.

It is never satisfactory to abandon any engineering work; nevertheless, it is the duty of the engineer to keep a careful watch, and forecast the ultimate expenditure of the work.

Taking all items into consideration, everything indicated that the ultimate cost of the dock would be exceptionally heavy, and, at the

same time, the probable return on the money expended through steamers utilizing the dock would be small; consequently, the Government was fortunate in being able to take advantage of the alternative scheme and utilize the work already done.

The latent resources of Western Australia, such as timber, mining, agriculture, dairying, wool, and fruit, are being rapidly developed by a progressive Government, and a further extension of the Harbor of Fremantle has been authorized.

Table 1 is a schedule of the rates of pay for the various classes of workmen engaged on the work, and may be of interest. The hours worked totaled 48 per week, and, in addition, there were 10½ paid holidays per annum.

TABLE 1.—Rates of Pay of Workmen.

Class.	Rate per day, in shillings.	Class.	Rate per day, in shillings.
Shift foreman	20	Engine drivers	12
Foreman laborer	14	Crane drivers	12
Sub-foreman laborer	11	Loco. drivers	12
Foreman carpenter	14	Loco. firemen	10
Foreman shipwright	14	Sawyer	11
Foreman fitter	15	Sawyer's laborer	9
Leading hand fitter	13	Wire splicer	10
Yard watchman	8	Sailormen	9½
Chainmen	9½ and 10	Plate-layers	9½
Storekeeper	12	Dam watchmen	9½
Storeman	10	Divers	20
Timekeepers	11 and 12	" tender	10
Clerks	10 and 11	" assistants	9½
Carpenters	12	" pumpers	9½
Blacksmiths	12½	Plumber	13
" striker	9½	" laborer	9½
Fitters	12	Timbermen	9½
" laborers	9	Foremen pile-drivers	11½
Firemen	10	Pile-drivers	9½
Trimmers	10	Laborers	9
Crane attendants	9½	Drillers	9

DISCUSSION

A. R. Archer, Assoc. M. Am. Soc. C. E. (by letter).—The work described by Mr. Ramsbotham is noteworthy by reason of the unusual conditions surrounding the driving of the steel sheet-piling, the varied character of the ground through which it was driven, and the extreme penetration obtained on the shore side, that is, at the back of the embayment for the pumping station. Mr. Archer.

In designing the bracing the author evidently desired to use a uniform size of timber throughout, with as little framing as possible, and, had it been necessary to brace the dam for the full depth of the diagram shown in Fig. 6, the spacing of the wales near the bottom would have been too close, as far as the strength of the steel skin was concerned. Within the limits to which excavation actually progressed, the spacing approximated very closely to that which the manufacturers of the piling recommended, but at greater depths this, necessarily, depends more on the character of the ground than on any theoretical considerations. The details of the bracing are in accordance with modern American practice, but the hardness of the jarrah wood used evidently permitted the omission of short foot-blocks at each end of the shores, in the lower tiers of bracing, such as are commonly used in the United States to reduce the span of the wales and prevent the end of the shore from cutting into the face of the waling, something which the writer has observed several times in relatively deep coffer-dams in which these foot-blocks were omitted. The bracing as a whole was designed for water pressure, but, from the conditions shown in the diagrams, that portion of the coffer-dam forming the shore side of the embayment for the pumping station was subjected to earth pressure, and it would be interesting to learn whether there was any indication of this exceeding the calculated water pressure. That something of the sort was anticipated is shown by the use of the concrete anchors with wire cable ties shown on Figs. 3 and 8.

The author has made several suggestions to the manufacturers of the steel sheet-piling, and in justice to the latter it should be stated that the pamphlet* published by them in 1912, covers most of the points to which attention has been drawn. A copy of this pamphlet is in the Library of the Society, and therefore available for reference by members.

In view of the large area to be unwatered—10 453 sq. ft.—and the severe leakage from the bottom, one might assume that leakage through the joints of the steel sheet-piling was not a serious factor; some information on this point would be of value. It is also to be assumed that the rapid corrosion of the steel by the discharge of sulphureted hydrogen would rapidly check any leaks which might exist.

* "Steel Sheet Piling: Tables and Data on the Properties and Uses of Sections."

Mr. Archer. The author should have the thanks of the entire Profession for his very careful description of grouting the bottom, as this is a subject on which but scant information can be gleaned from published engineering works.

Mr. Bellamy. HERBERT E. BELLAMY, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read Mr. Ramsbotham's paper with much interest, and is glad that a matter of such importance has been presented before the Society.

As the writer has had considerable experience with jarrah timber for walings and shores, as referred to by the author, the following remarks may not be considered out of place.

Jarrah (*Eucalyptus marginata*) is one of the most valuable of Western Australian timbers, and one of the best in the Australian Colonies. It is, perhaps, more widely known to engineers outside Australia than any other timber grown there. It is exceedingly durable, whether exposed so as to be alternately wet and dry, or used as piles in sea-water or damp ground. Its weight varies from 56 to 68 lb. per cu. ft. Recent tests of this timber afford the following information:

Transverse strength per square inch	1 800 lb.
Tension: direct cohesion on 1 sq. in.	2 940 "
Crushing strain on 2-in. cubes	3.198 tons.

It is a singular fact, however, that the majority of engineers throughout Australia do not view with favor the use of hard woods for walings.

One particular instance, in which absolutely nothing but jarrah was used for timbering, was in the excavation for the main pumping station executed under the writer's supervision in connection with the Colombo (Ceylon) Main Drainage Works, for which works, Messrs. James Mansergh and Sons were the Consulting Engineers. All the walings and struts were 12 by 12-in. and fixed at 4-ft. centers. Fig. 11 illustrates the system adopted. All sets were independent and in 20-ft. lengths, so as to facilitate the easy striking of the timbers as the masonry arose from the bottom.

When the excavation was down to formation level and all timbers were in, heavy monsoonal rains set in, and the trench was allowed to be flooded for several weeks. After the rains had ceased, pulsometer pumps were fixed, and the water was pumped out so that the bottom concrete could be put in. On several subsequent occasions, the trench was flooded and pumped out again. This alternate wetting and drying would naturally tax any timber, but the writer can state with every confidence that every waling and strut removed from this excavation after being in use for about 12 months, was in practically the same good condition as when first put in. Indeed, the majority of the

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FIG. 11.—WIDE TRENCH, AT COLOMBO, CEYLON, SUPPORTED BY 12 BY 12-IN. WALING AND

timbers taken out were sawn into 12 by 6-in. and 12 by 3-in. scantlings and used in other parts of the work. Mr. Bellamy.

The writer has also used jarrah extensively for side-trees, head-trees, and sills in tunnels, which have been flooded on several occasions, and not in a single instance was there failure or disappointment.

Steel sheet-piling has been used so extensively and successfully, and in so many different classes of work, that the author's elaborate description calls for no further comment.

JOSHUA FIELDEN RAMSBOTHAM, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Archer has referred to the penetration of the piles obtained on the shore side of the pumping station, and as it may be of interest the writer submits Fig. 12, showing a pile which failed; altogether, there were eight piles which went astray. It is only fair to state that the rock in the locality was as hard as bell-metal, and the behavior of the piles in no wise contributed to the abandonment of the work. The writer would again emphasize the advisability of not forcing the piles until soft ground has been found ahead of piles which have encountered a hard patch. Mr. Ramsbotham.

Mr. Archer is quite correct in his surmise that the hard wood (jarrah) permitted the omission of short foot-blocks.

For the information of the members the writer has forwarded, for the Library of the Society, two books* which give all details about the various timbers of Western Australia, and he ventures to suggest that that country has been singularly blessed by Providence in this respect.

No doubt Mr. Archer has observed that a larger factor of safety was allowed on the pumping station shores; this was because of the additional weight due to earth pressure and any surcharge from cranes, etc., and also on account of their length.

As regards the suggestions to the manufacturers, the writer begs to state that he gave freely any benefits of his experience at the time to Mr. U. V. Burke, Agent for the United States Steel Products Company, and also Consul for the United States of America, and Mr. Burke reciprocated with all particulars from the manufacturers; by this means the Engineering Profession is best served.

Mr. Archer is quite correct in assuming that there were no leaks in the joints; the skin was quite tight, and this tightness was obtained by the following method: Steel rods, 20 ft. long and $\frac{1}{4}$ in. in diameter, were procured, their lower ends were made an inverted **V** shape, and their upper ends a wedge shape with a small hole drilled through. A loop was made in one end of a strand of Italian gasket, the diameter not being increased. The inverted **V** end of a rod was in-

* "Western Australian Timber Tests, 1906: The Physical Characteristics of the Hardwoods of Western Australia," by G. A. Julius. Second Edition, 1908; and "Notes *re* Timbers of Western Australia Suitable for Railways, Engineering Works, and Constructional Purposes Generally," Second Edition, 1908.

Mr. Ramsbotham.

serted in this loop and the rod lowered in the space between the male and female piles by a fine string put through the eye. This was repeated by lowering another rod on a separate string with an additional length of gasket, until the entire length of joint was traversed. The gasket made a complete seal and, of course, the rods were removed.

In the first instance Oregon pine fillets were inserted when driving the pile, but there is no exaggeration in stating that they were completely eaten up by the teredo in one month.

Any small trickles can be cured instantly by throwing in a mixture of saw-dust, ashes, and horse manure.

The writer is glad to hear that Mr. Bellamy has used jarrah with success, and this opportunity is taken to supplement his remarks on this head.

Jarrah (*Eucalyptus marginata*) trees average 90 or 100 ft. in height and from 2 to 3 ft. in diameter at the base. The wood is very hard and dense, weighing when seasoned 60 lb. per cu. ft. It is used economically for railway sleepers, jetties, bridges, marine and engineering works of all kinds, building construction, flooring, wood blocking, boat building, fencing, furniture, etc.

Its strength is as follows: Shearing, 2 010 lb. per sq. in.; end compression, 9 050 lb. per sq. in.; cross-bending, 15 000 lb. per sq. in.; tension, 15 500 lb. per sq. in. Additional information regarding this wood may be found in the books previously quoted.

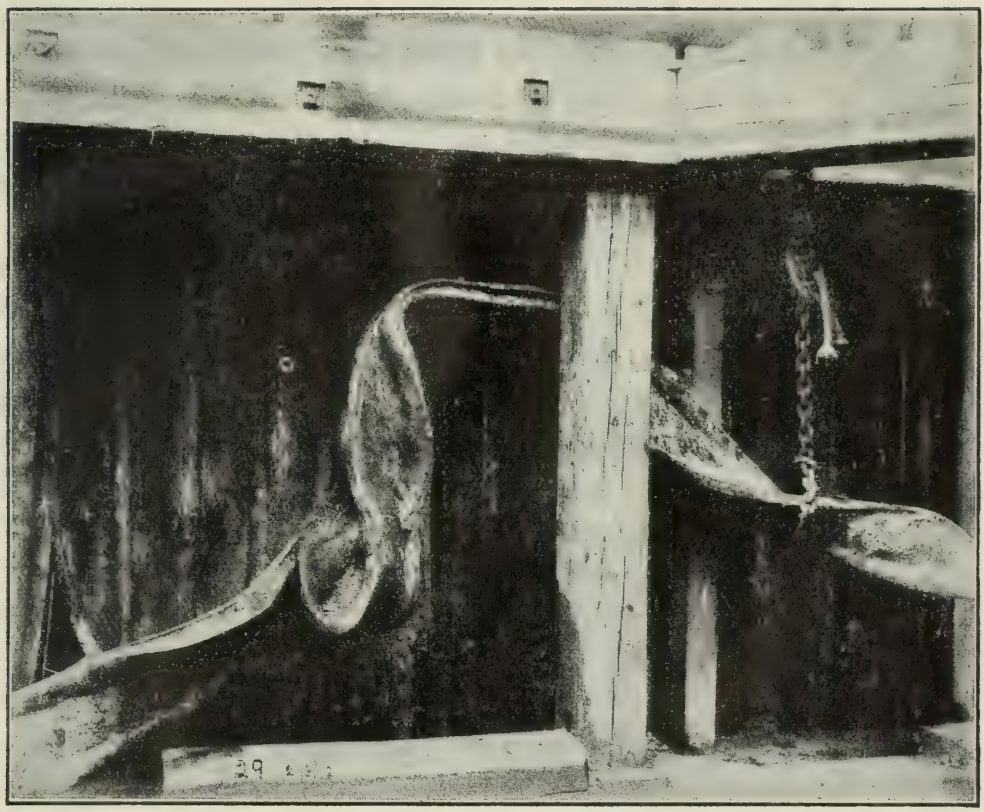


FIG. 12.—STEEL PILES DRIVEN ON WEST SIDE OF PUMPING STATION, NORTH WALL, FREMANTLE GRAVING DOCK.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1278

TIDAL PHENOMENA IN THE HARBOR OF NEW YORK.*

By H. de B. Parsons, M. Am. Soc. C. E.

With Discussion by Messrs. Allen Hazen, T. Kennard Thomson, James Owen, Kenneth Allen, and H. de B. Parsons.

The Harbor.

For the purpose of this paper, the harbor of New York is taken to include the Lower Bay, the Upper Bay, the Hudson River to Mount St. Vincent, the East River to Throgs Neck, the Harlem River, Newark Bay, the Kill van Kull, the Arthur Kill, and Jamaica Bay. Each of these divisions can be considered a distinct branch, although the tidal phenomena, taken as a connected whole, are affected by the characteristics of each.

A map of the harbor, Fig. 1, shows the positions of these different parts and their relations to each other.

It is difficult to describe the tidal actions clearly and concisely because the movements of the water in all parts of the harbor are not synchronous. This is due to the following causes:

1.—The harbor is so extended that there is a considerable difference in time between the periods of high water at various points;

2.—The variation in the range of tides;

3.—Some of the currents being produced by progressive tidal wave motion, some by hydraulic conditions (that is, difference in water levels), and some by the interference of two distinct tidal waves;

* Presented at the meeting of May 7th, 1913.

4.—The variable flow of fresh-water into the harbor from the rivers (that is, seasonal variation);

5.—The under-run, caused by the inertia of the flowing stream, the difference in specific gravity between sea-water and land-water, and possibly by the configuration of the harbor bottom.

Average conditions exist only occasionally. Variations in tidal range at any station are caused by the relative positions of the moon and sun, and by other factors. The winds also have a great temporary influence on the rise and fall of the tides. Outside of temporary influences, such as wind, the regular changes are periodic; therefore, the difficulty of describing the phenomena will be apparent, because the conditions which exist on one day will not exist on the days preceding or following. As the general phenomena will depend closely on average conditions, the tidal phenomena described herein will be based on mean conditions of tidal range, velocities, and volumes of tidal flow.

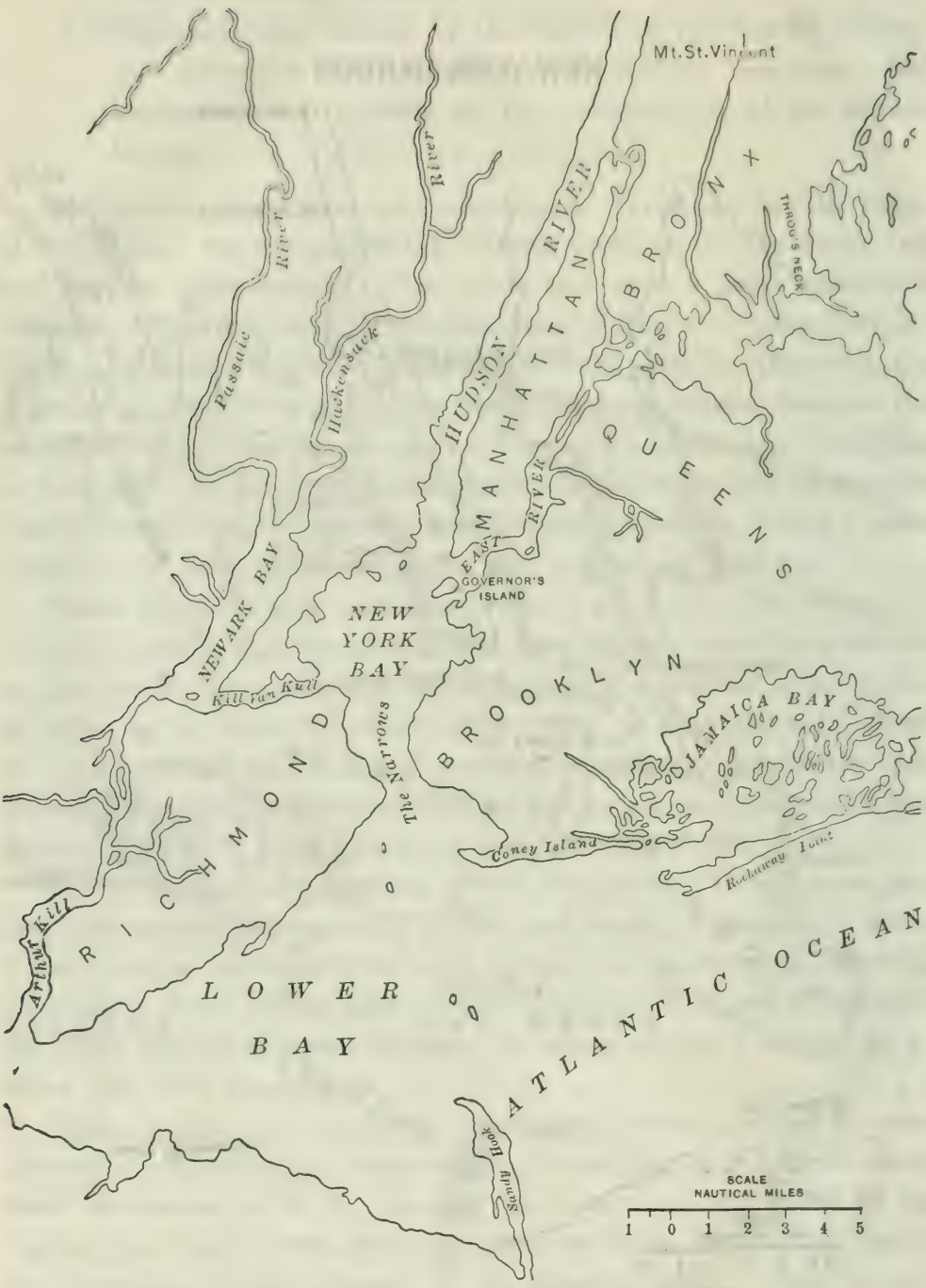
Where velocities have been observed on a given date they have been corrected to what they would have been had the mean conditions existed on that date. The method for the correction of velocities is the multiplication of the observed velocity by the ratio of the mean tidal range to the tidal range on the day in question, in accordance with the conditions given for the different parts of the harbor, as stated under the heading "Effect of Tidal Range on Velocity," page 1996.

The water gradually shallows toward the shores. If the water surface was lowered, the boundary of the land would be changed in form. These changes are interesting and instructive, in studying the tidal phenomena of the harbor, and are shown in Figs. 2, 3, and 4, in which the water surface is drawn at mean low water, and at 6 ft. and 12 ft. below this level, respectively.

These figures show that, of the aggregate of water surfaces at mean low water, only about 58% has a depth of more than 12 ft. If the water level was lowered to 24 ft., the only water surfaces left would be the Hudson and East Rivers, which uniting just below the Battery, would flow through the main channel to the Atlantic Ocean. That is, only the important channels would remain. Taking the harbor divisions as defined in Appendix A, the water surfaces at different planes would be approximately as given in Table 1.

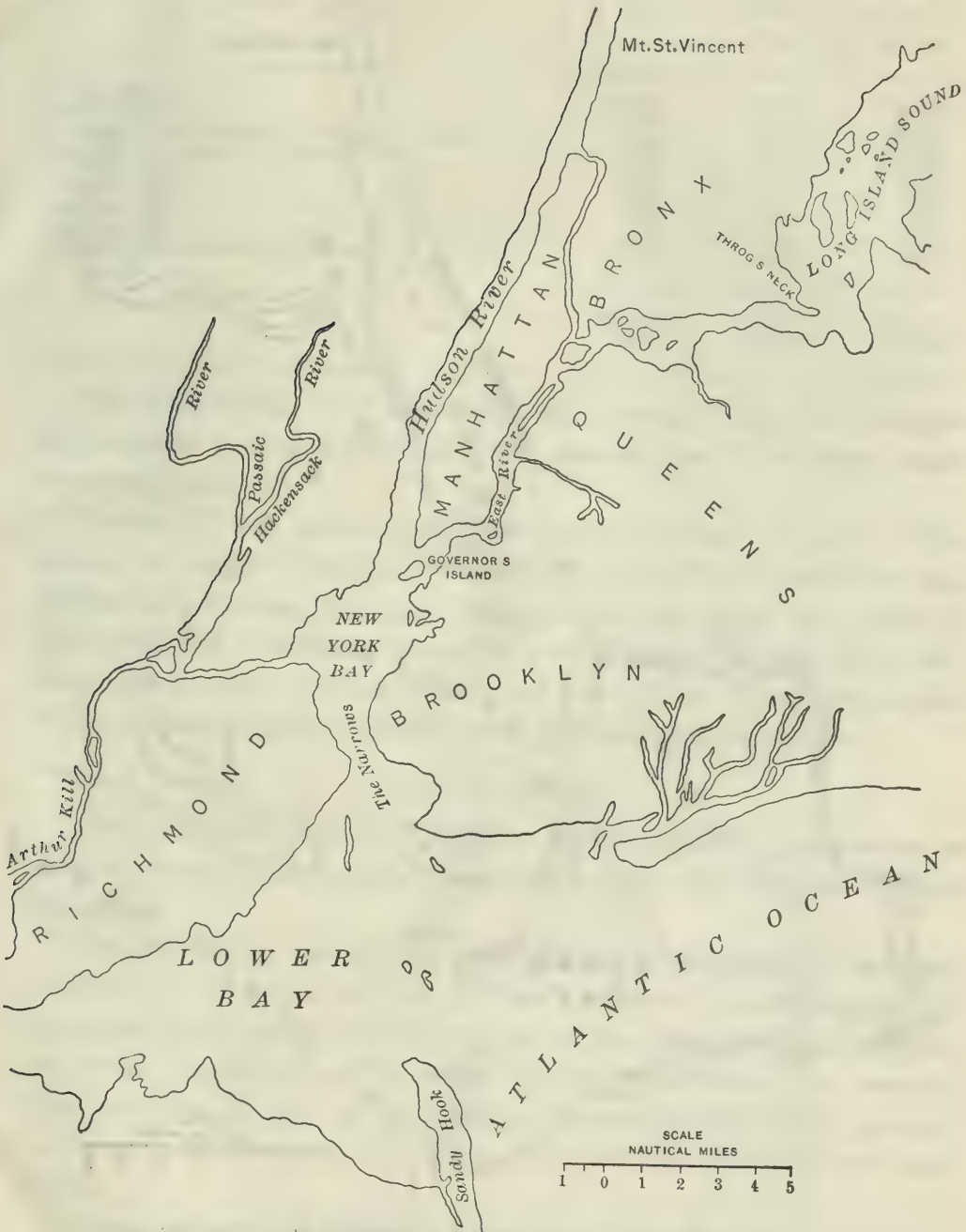
NEW YORK HARBOR
Tarrytown
Mt.St.Vincent
Ft.Washington Point
Passaic River
Hackensack River
HUDSON RIVER
MANHATTAN
Harlem River
BRONX
LONG ISLAND SOUND
Throgs Neck
Willets Point
Hell Gate
BLACKWELLS ISLAND
East River
QUEENS
Passaic Light
NEWARK BAY
GOVERNORS ISLAND
NEW YORK BAY
Robbins Reef
Kill van Kull
The Narrows
BROOKLYN
JAMAICA BAY
Ft.Wadsworth
RICHMOND
Arthur Kill
West Bank
LOWER BAY
OCEAN
ATLANTIC
Great Beds
Sandy Hook
Ambrose Light
SCALE
NAUTICAL MILES
½ 0 1 2 3 4 5

FIG. 1.



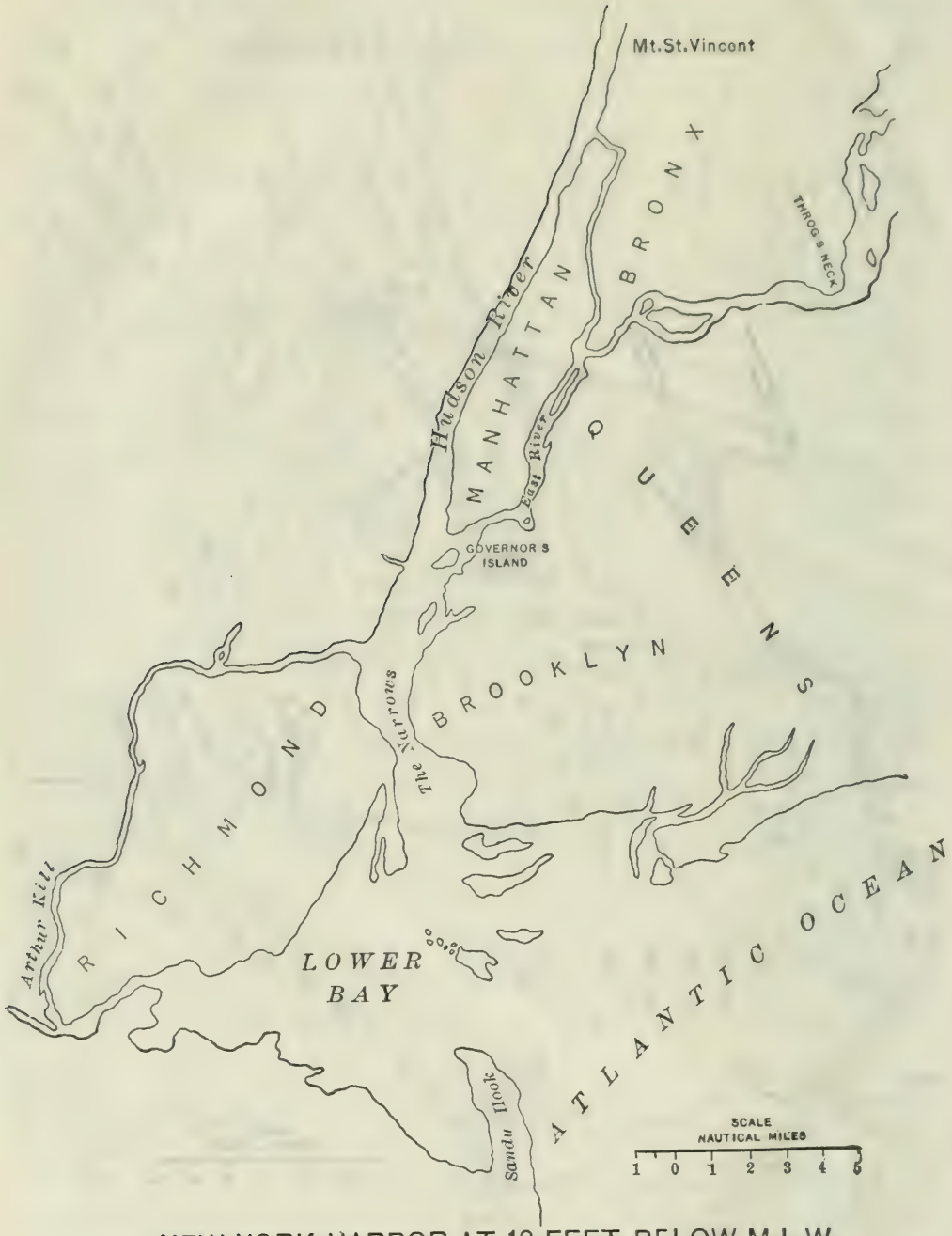
NEW YORK HARBOR AT MEAN LOW WATER

FIG. 2.



NEW YORK HARBOR AT 6 FEET BELOW M.L.W.

FIG. 3.



NEW YORK HARBOR AT 12 FEET BELOW M.L.W.

FIG. 4.

TABLE 1.—Areas of Water Surfaces, in Square Miles.

Part of harbor.	At M. L. W.	At Depths Below M. L. W.	
		6 ft.	12 ft.
Lower Bay	124.30	106.20	83.60
Upper Bay	20.74	14.88	12.36
Hudson River, Battery to Mt. St. Vincent	14.49	13.73	12.88
East River, Battery to Throgs Neck	15.20	10.30	8.54
Harlem River	0.59	0.47	0.38
Newark Bay	8.35	3.28	0.64
Kill van Kull	1.12	0.97	0.88
Arthur Kill	4.63	3.34	2.31
Jamaica Bay, excluding islands	21.61	4.40	2.20
	211.03	157.57	123.79
	100%	74.6%	58.6%

The phenomena of "under-run" and "inertia" are well confined within areas having a depth of water greater than 12 ft. below mean low-water level.

Profiles of the harbor bottom are shown in Figs. 5 and 6. The former is a profile through the Lower Bay, Upper Bay, East River, and Hudson River, along the line of the channel; the latter is a profile of average depths of sections across the stream along the same line. Between the Atlantic and the Sound entrances of the harbor, the depths of the sections in general tend to vary inversely as their widths.

Tidal Currents.

The normal currents in the harbor are produced by progressive or periodic waves, by hydraulic conditions, or by a combination of these two causes.

The progressive wave is the natural motion, due mainly to the attractions of the moon and sun. When the wave is unobstructed, the maximum flood and ebb velocities occur at about the times of high and low water, respectively. The profile of a true tidal wave is a sinusoid. The velocities, as well as the heights (stages) of the water, are proportional to the ordinates of a sinusoid the abscissas of which have time values.

When hydraulic conditions obtain, a current flows, on account of the difference in head which exists temporarily between the bodies of water connected. In this case the maximum velocities occur at some time between local high and low waters, depending on the time of greatest hydraulic slope and the characteristics of the channel.

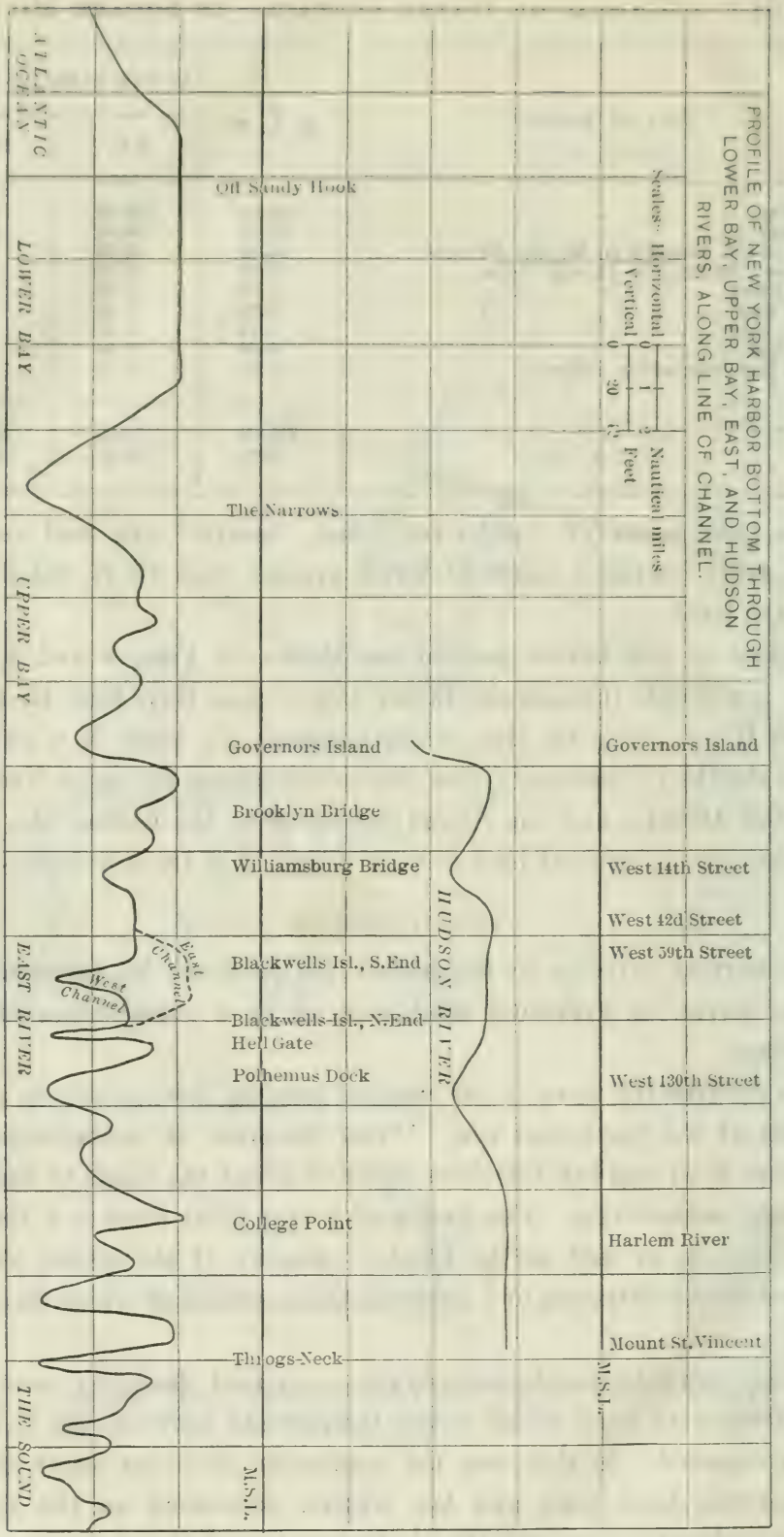

PROFILE OF NEW YORK HARBOR BOTTOM THROUGH
LOWER BAY, UPPER BAY, EAST, AND HUDSON
RIVERS, ALONG LINE OF CHANNEL.
Scales: Horizontal 0 1 3 Nautical miles
Vertical 0 20 Feet
ATLANTIC OCEAN
LOWER BAY
UPPER BAY
EAST RIVER
HUDSON RIVER
THE SOUND
Off Sandy Hook
The Narrows
Governors Island
Brooklyn Bridge
Williamsburg Bridge
Blackwells Isl., S.End
Blackwells Isl., N.End
Hell Gate
Pothemus Dock
College Point
Throgs Neck
East Channel
West Channel
M.S.L.
Governors Island
West 14th Street
West 42d Street
West 59th Street
West 130th Street
Harlem River
Mount St. Vincent
M.S.L.

FIG. 5.

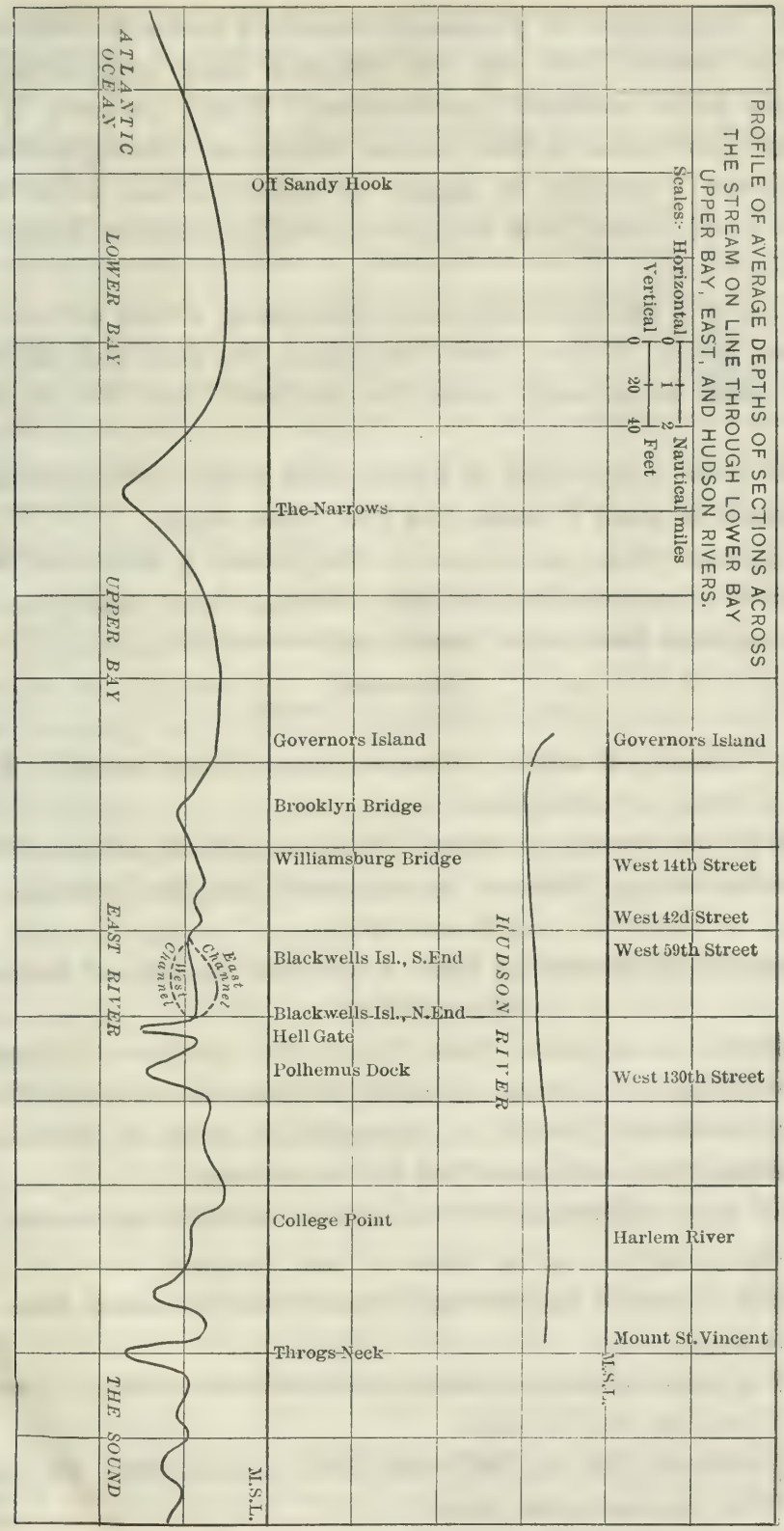

PROFILE OF AVERAGE DEPTHS OF SECTIONS ACROSS THE STREAM ON LINE THROUGH LOWER BAY UPPER BAY, EAST, AND HUDSON RIVERS.
Scales:- Horizontal 0 1 2 Nautical miles
Vertical 0 20 40 Feet
ATLANTIC OCEAN
LOWER BAY
UPPER BAY
EAST RIVER
THE SOUND
HUDSON RIVER
Off Sandy Hook
The Narrows
Governors Island
Brooklyn Bridge
Williamsburg Bridge
Blackwells Isl., S.End
Blackwells Isl., N.End
Hell Gate
Polhemus Dock
College Point
Throgs Neck
East Channel
West Channel
M.S.L.
Governors Island
West 14th Street
West 42d Street
West 59th Street
West 130th Street
Harlem River
Mount St. Vincent
M.S.L.

FIG. 6.

When a combination of progressive wave and hydraulic conditions obtains, the greatest flood and ebb velocities occur irregularly, depending on which condition predominates. When the wave is restricted by interference or other causes, the curves (having ordinates which represent velocities or heights of water surface at different periods of time) depart from true sine curves, in amounts depending on the restriction.

The time of a complete tidal period, comprising a flood and an ebb, is 12 lunar hours. Under normal conditions, the flood and ebb flow during 6 lunar hours each, except for the small time lost at slack waters.

A lunar hour is equivalent to 1.03505 solar hours, or 6 lunar hours are equivalent to about 6 hours, 12.6 min., solar time.

In a channel where the current follows closely a true tidal wave action, which is harmonic in character, a floating object will be carried, during a complete flood or ebb period, a distance of

$$\frac{2}{\pi} \times A \times \frac{12.4206}{2} \text{ knots,}$$

or nearly $4A$ knots, in which A denotes the maximum velocity of the current, in knots per solar hour.

Similarly, the velocity at which a floating particle will be carried by the current at any time can be estimated from the formula,

$$A \text{ cos. } 30t,$$

where t denotes the number of lunar hours after the time of strength; or

$$A \text{ cos. } 28.98t,$$

where t denotes the number of solar hours after the time of strength.

By computing and plotting a number of these velocities at different stations in a selected locality, it is possible to make an estimate of how a floating object will be drifted by the currents.

There is some difference, however, due to the following causes:

1.—The under-run at the times of tidal changes;

2.—The inertia of the flowing stream when it changes from ebb to flood;

3.—The shore currents not being the same as the channel currents;

4.—Variations due to wind;

5.—Variations due to land-water flow coming down the rivers from drainage areas above;

6.—Variations in tidal ranges.

Owing to variations in channel sections and directions, which cause side currents, it is not safe to predict the course which a floating particle will take. It is safer to rely on observations with floats, many of which have been made by the Metropolitan Sewerage Commission,* and will be referred to later.

It must be remembered that the surface velocities do not readily indicate the mean velocities of tidal streams. Although the surface particles may have a known velocity in one direction, the velocity and direction of the particles beneath the surface often differ materially.

To determine accurately the mean velocity of a tidal current, it would be necessary to take synchronous observations at many points across a stream and at many points in depth below the surface. Such observations should extend over a considerable period of time in order to obtain fair averages.

Interference Tides.

When separate tidal waves enter a strait from opposite ends, there results in the strait an "interference" tide. This is not necessarily the algebraic sum of the two separate tides. In other words, when the two waves meet and overlap, the result of the overlap is not necessarily a superposition of one wave upon the other.

The actual tidal rise or fall above or below mean sea level, other things being equal and not acting as disturbing factors, at any hour of the tidal interval, is equal to a coefficient multiplied by the algebraic sum of the rises or falls of the two tides.† The difference in times between high and low water at each end of the strait is an important factor.

Interference tides occur in the harbor because separate tidal waves enter, one from the sea past Sandy Hook and the other from Long Island Sound past Willets Point. These separate tidal waves are not synchronous, that is, their periods of high or low water do not occur at the same time.

The Sound tide‡ (through Hell Gate) is distinctly traced to Governors Island, where, although much reduced in range, it is frequently found to affect the observed tide, mostly in the way of reducing the

* Report, April 30th, 1910.

† Henry Mitchell, Chief Physical Hydrographer, used 0.65 for the coefficient in computing the composite tide in the East River. Thus, the predicted tide was estimated by taking 0.65 times the algebraic sum of the difference in heights of the tides measured from mean sea level. U. S. Coast and Geodetic Survey, 1867. Appendix No. 13.

‡ U. S. Coast and Geodetic Survey, 1866. Appendix No. 6.

rise and fall. The southern tide from The Narrows meets a small propagation of the Sound tide, the lunar interval of which differs nearly 6 hours; the result is a reduction in range. This is the principal reason why the tidal range at Governors Island is less than that at Sandy Hook. The tidal range is also reduced by the expansion of The Narrows' tidal wave in Upper New York Bay.*

The East River is a strait, at one end of which the tidal range is 4.4 ft., and at the other 7.2 ft. Where the high range occurs (Willets Point) the tidal wave enters 3 hours, 5 min. later than the tidal wave at the other end (Governors Island). The two tides meet and cross or overlap each other at Hell Gate; and as they differ in times and heights, they cause contrasts of water elevations which call into existence violent currents.

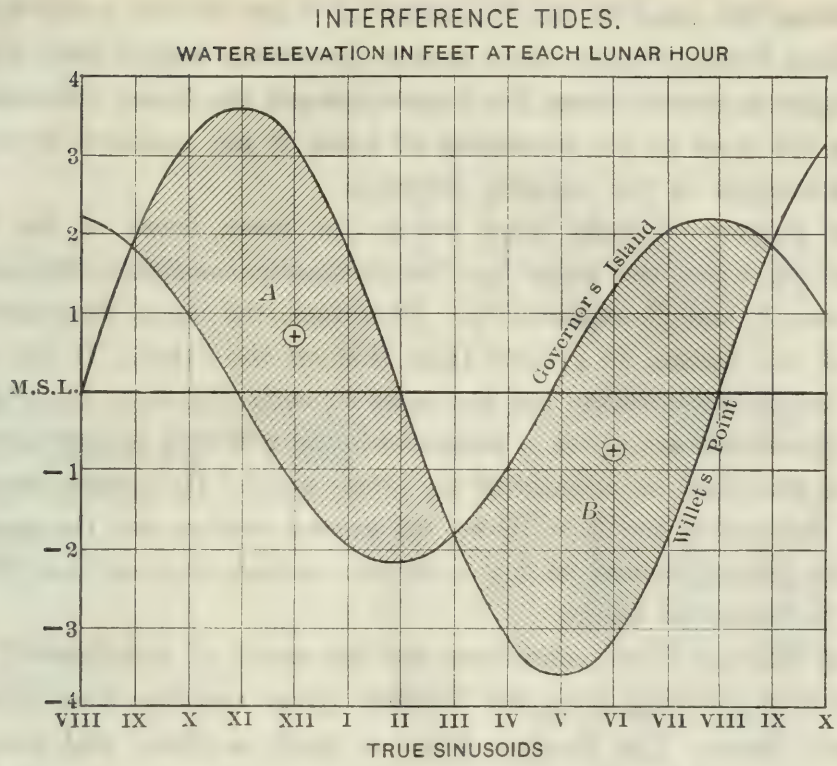
The volume of water flowing through the strait in one direction is not necessarily equal to that flowing in the opposite direction, but depends on the relative times when each wave enters its respective end of the strait, as well as on the total range at each end.

If one wave enters later than the other, and has a greater range, the mean height of the water when the current is flowing one way will be higher than the mean height when the current is flowing contrariwise. Consequently, the sectional area of the stream in one case will be greater than in the other, and a larger volume of water is likely to pass during a tidal variation.

Suppose curves for the waves entering at each end of a strait be drawn, and then that the two curves be superimposed so that the distance between the peaks will represent the difference in time between high waters at each end, as shown in Fig. 7. The upper curves are true sinusoids, representing the mean ranges of tide at each end of the East River, namely Governors Island and Willets Point. The lower curves show the East River conditions for one set of observations, as recorded in the Report of the U. S. Coast and Geodetic Survey for 1888, Appendix No. 9.

Lunes† will be formed at *A* and *B*. When the curves are true sinusoids, their areas are equal; but, under actual tidal conditions, their areas are only approximately equal. These lunes are unbalanced, for the center of one is higher than that of the other, as shown by the crosses on Fig. 7.

*U. S. Coast and Geodetic Survey, 1867, Appendix No. 13.
†U. S. Coast and Geodetic Survey, 1886, Appendix No. 13.


INTERFERENCE TIDES.
WATER ELEVATION IN FEET AT EACH LUNAR HOUR
4
3
2
1
M.S.L.
−1
−2
−3
−4
A
B
Governor's Island
Willet's Point
VIII IX X XI XII I II III IV V VI VII VIII IX X
TRUE SINUSOIDS

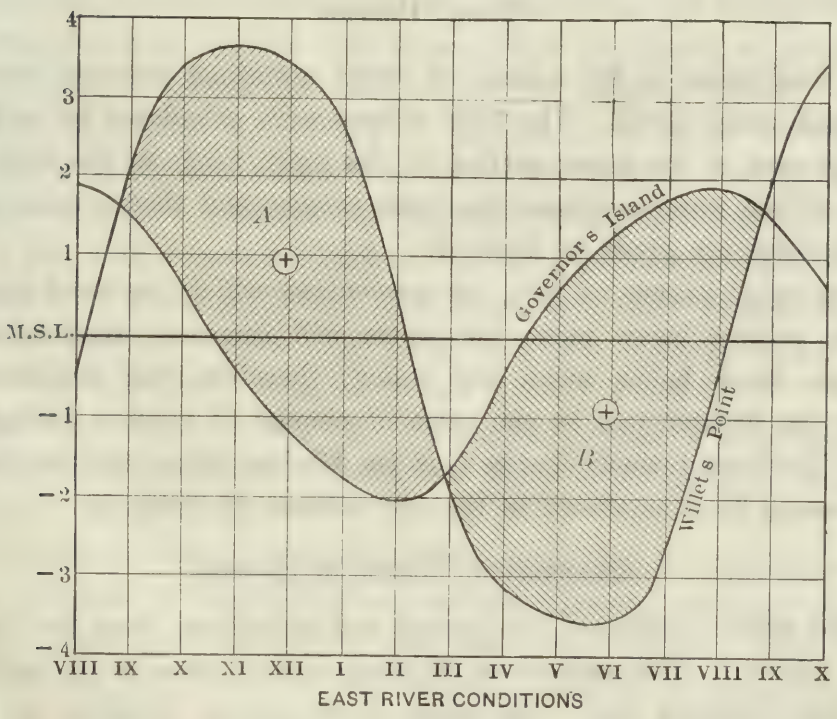

4
3
2
1
M.S.L.
−1
−2
−3
−4
A
B
Governor's Island
Willet's Point
VIII IX X XI XII I II III IV V VI VII VIII IX X
EAST RIVER CONDITIONS

FIG. 7.

During the time of the formation of Lune *A*, the water surface at Willets Point is higher than that at Governors Island, thus tending to produce a current from the former toward the latter. Conversely, during the time of the formation of Lune *B* the tendency is to produce a current in the opposite direction.

The centers of these lunes are at the mean levels of the water during the times the water has the hydraulic conditions represented by Lunes *A* and *B*, respectively. Therefore, the mean cross-sectional area of one stream is greater than that of the other. If the mean velocities of the currents are the same in each direction, then, as the hydraulic slopes are equal, a greater volume will flow in one direction than in the other, on account of the larger area of the stream section.

In the case of the East River, the greater section and the tendency toward a greater volume of flow is for the current running from Willets Point to Governors Island.

The Harlem River conditions are the result of interference tides, with waves entering from the Hudson River and the East River at different times. The Harlem River is really a strait, and what has been said about the East River would also apply in a modified way to it.

Tidal Prisms.

A tidal prism is the volume of water contained between low-water and high-water levels. The tidal prisms were calculated by multiplying the area of the water surface by the mean range of the tide. The areas of the water surfaces, the tidal ranges, and the volumes of the tidal prisms are given in Table 2.

For equal ranges of tide, the percentage which the tidal prism is of the volume below mean low water will increase inversely as the average depth below mean low water; therefore, the shallower the water the greater will be the ratio of change of volume during each tide. The least ratio of change is in the Hudson River, and the greatest in Newark Bay, as shown in the last column of Table 2.

Quantity of Water in Harbor.

The water surface of the harbor was calculated from the Government charts.* This surface is 65.12 sq. miles, exclusive of the Lower Bay and Jamaica Bay. The areas of the several divisions are given

*The boundaries of the harbor divisions are given in Appendix A.

in Table 2. The area of the Lower Bay is 124.3 sq. miles, and of Jamaica Bay 21.61 sq. miles.

TABLE 2.—Areas, Depths, Tidal Ranges, Volumes, and Tidal Prisms.

Part of harbor.	Area, in square miles.	Average depth, in feet.	Mean tidal range, in feet.	Volume below M. L. W., in millions of cubic feet.	Tidal prism, in millions of cubic feet.	Percentage.*
Upper Bay	20.74	22.4	4.4	12 970	2 541	19.6
Hudson River, Battery to Mt. St. Vincent.	14.49	30.7	4.2	12 330	1 697	13.7
East River, Battery to East 88th Street..	3.50	31.7	4.7	3 091	459	14.8
East River, East 88th Street to Old Ferry Point	9.12	22.3	6.2	5 680	1 575	27.7
East River, Old Ferry Point to Throgs Neck	2.58	38.8	7.1	2 791	511	18.3
Harlem River	0.59	11.3	5.3	189	88.2	46.6
Newark Bay	8.35	6.6	4.6	1 542	1 071	69.5
Kill van Kull	1.12	23.4	4.8	728	149.8	20.6
Arthur Kill	4.63	13.1	5.4	1 690	697	41.2
Totals	65.12	....	...	41 011	8 789.0	....

* Ratio of tidal prism to volumes below M. L. W. expressed as a percentage.

An estimate of the volumes of water below mean low water was made in the following manner: For the Upper Bay, Newark Bay, and the East River between East 88th Street, Manhattan, and Throgs Neck, the chart areas were divided into squares, and the depths as given on the chart were averaged for each square or fraction thereof. The shore lines were taken as marked on the charts for mean low water. The volumes under each square were found and added. For the Hudson River, East River south of 88th Street, Harlem River, Arthur Kill, and Kill van Kull, cross-sections were drawn at regular intervals, from information on Government charts of large scale, and cross-sectional areas were obtained. From these areas and their distances apart the volumes were calculated. The results are given in Table 2.

At any hour, there is more water in the harbor than is given by the sum of these figures, because mean low water does not occur at the same hour in all its parts.

The volume of water is variable from hour to hour, and the approximate mean value is nearly the sum of the quantity of water below mean low water and one-half of the mean tidal prism. The tidal prisms are given in Table 2. The average quantity of water, therefore, is not far from the sum of 41 011 000 000 and 4 394 500 000, or 45 405 500 000 cu. ft., exclusive of the Lower Bay and Jamaica Bay.

Tidal Ranges.

The tidal ranges, or the rise and fall of the tides, are not the same at all points in the harbor; also, the hours at which high and low water occur at different points vary considerably. These statistics are given in Table 3.

TABLE 3.—Mean Range of Tides, and Times of High and Low Water.*

Station.	Mean tidal range, in feet.	Times.†		Differences in Times.‡	
		H. W.	L. W.	H. W.	L. W.
Sandy Hook (Horseshoe)	4.7	7.35	1.27	—0.29	—0.38
Canarsie, Jamaica Bay	4.2	8.34	2.35	0.30	0.30
Tottenville, Arthur Kill	5.6	7.55	1.59	—0.09	—0.06
Shooters Island, Newark Bay	4.6	8.20	2.28	0.17	0.23
Passaic Light, Newark Bay	4.7	8.41	2.59	0.38	0.54
Fort Hamilton, Narrows	4.6	7.41	1.38	—0.23	—0.28
Governors Island	4.4	8.04	2.05	0.00	0.00
Blackwells Island Light	5.3	9.54	3.39	1.50	1.33
Throgs Neck	7.3	11.09	5.14	3.05	3.09
Spuyten Duyvil	4.0	8.49	2.51	0.45	0.46

* Tide Tables, U. S. Coast and Geodetic Survey.

† Solar time, in hours and minutes, after transit of moon.

‡ From that at Governors Island, in hours and minutes.

Current Velocities.

The current velocities are variable throughout the harbor, and are dependent on the conditions existing at the time of observation. Under normal conditions, the coefficient which expresses the relation between the swiftest surface thread of a current and the mean cross-sectional velocity is probably fairly constant for any one section, although it is affected by inertia and the reversal of the currents due to change of direction, which reversal does not take place simultaneously throughout a section.

Under normal conditions, the swiftest surface velocities (or the velocities just under the surface) of the tidal currents, when flowing at strength, are given in Table 4, in knots per solar hour, as well as the normal duration of surface currents, in lunar hours. These velocities and durations were obtained from records of the Coast and Geodetic Survey and from float observations made by the Metropolitan Sewerage Commission.

The mean surface velocity of the swiftest thread is approximately $\frac{2}{\pi}$ times the velocity at strength, as the wave formation is harmonic in character.

TABLE 4.—Normal Swiftest Surface Velocities at Strength, in Knots; and Normal Duration of Surface Currents, in Lunar Hours.

Station.	Velocity at Strength.		Duration,* in Lunar Hours.	
	Ebb.	Flood.	Ebb.	Flood.
The Narrows	2.1	1.8	6.7	5.3
Hudson River, off 39th Street	3.0	2.0	6.7	5.3
East River, Brooklyn Bridge	3.8	3.6	6.6	5.4
" " off 11th Street	3.0	2.9	6.1	5.9
" " off 31st Street	2.9	2.6	6.0	6.0
" " at Hell Gate	4.8	4.7	5.9	6.1
" " off Old Ferry Point	1.3	1.3	5.8	6.2
Kill van Kull, off Port Richmond	2.2	1.9	6.3	5.7
" " " 0.1 mile S. of Bergen Point	2.0	1.8	6.4	5.6
Harlem River, off 114th Street	1.0	1.0	5.9	6.1
" " 600 ft. N. of High Bridge	1.9	1.8	6.0	6.0

*The durations of slack-water are not the same in all parts of the harbor, nor does the current change at the same time in all parts of a section. The slack-water periods are included in the duration of the currents, which add to 12 lunar hours.

Some observations of the velocities of the currents at different depths have been made in a few sections in the harbor.* They were made at one or at only a few stations in a section, and the records are not sufficiently full to be of material value in determining the mean cross-sectional velocity, or the value of the coefficient expressing the relation between the velocity of the swiftest surface thread and the mean velocity of the section. They show, however, that the currents do not reverse simultaneously in a section, and also that the period of time occupied by a complete change of current is not the same for ebb to flood as for flood to ebb. Generally speaking, the period is longer in the former case and shorter in the latter.

The surface of some parts of the bottom of the harbor is much freer from fine silt and light deposits than others. It also appears that when the velocities exceed $1\frac{1}{2}$ ft. per sec., the finer and lighter materials are washed away and deposited in the deeper or quieter parts.

*Reports of U. S. Coast and Geodetic Survey.

Effect of Tidal Range on Velocity.

In the East River the tidal currents are nearly hydraulic, that is, they flow from the body having temporarily the higher water-surface level to the one having temporarily the lower. In other words, the flow is caused by the difference in height which temporarily exists between the bodies connected. In consequence, the velocities in the East River vary closely as the square root of the range of tide.

In the Hudson River the tidal currents are due chiefly to the progressive wave motion, as is shown by the fact that the greatest flood and ebb velocities occur at nearly the times of local high and low waters. In the Hudson, therefore, the velocities vary directly as the range of tide.

In the Kill van Kull and in the Arthur Kill the tidal currents are nearly hydraulic, and in them, as in the East River, the velocities vary closely as the square root of the range of tide.

In The Narrows the tidal currents are partly hydraulic and partly due to the progressive wave motion. In consequence, the velocities there vary approximately midway between the square root of and directly as the range of tide.

In the Harlem River the tidal currents are nearly hydraulic, and are due to a temporary difference in water level in the East River and in the Hudson. Therefore, the velocities vary approximately as the square root of the range of tide.

In Rockaway Inlet, joining Jamaica Bay and the Atlantic Ocean, the currents are hydraulic. The velocities, therefore, vary as the square root of the range of tide, the greatest velocities occurring about 3 hours before high or low water.

Currents at Each Lunar Hour.

The currents which exist in the harbor at each lunar hour of a tidal cycle are given below. The directions of the currents were obtained chiefly from the float experiments of the Metropolitan Sewerage Commission, and were checked from the curves shown on Fig. 8. Other observations were also made, for purposes of verification. The construction of the curves in Fig. 8 is explained on page 2056.

I Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; into the Upper Bay, through the Kill

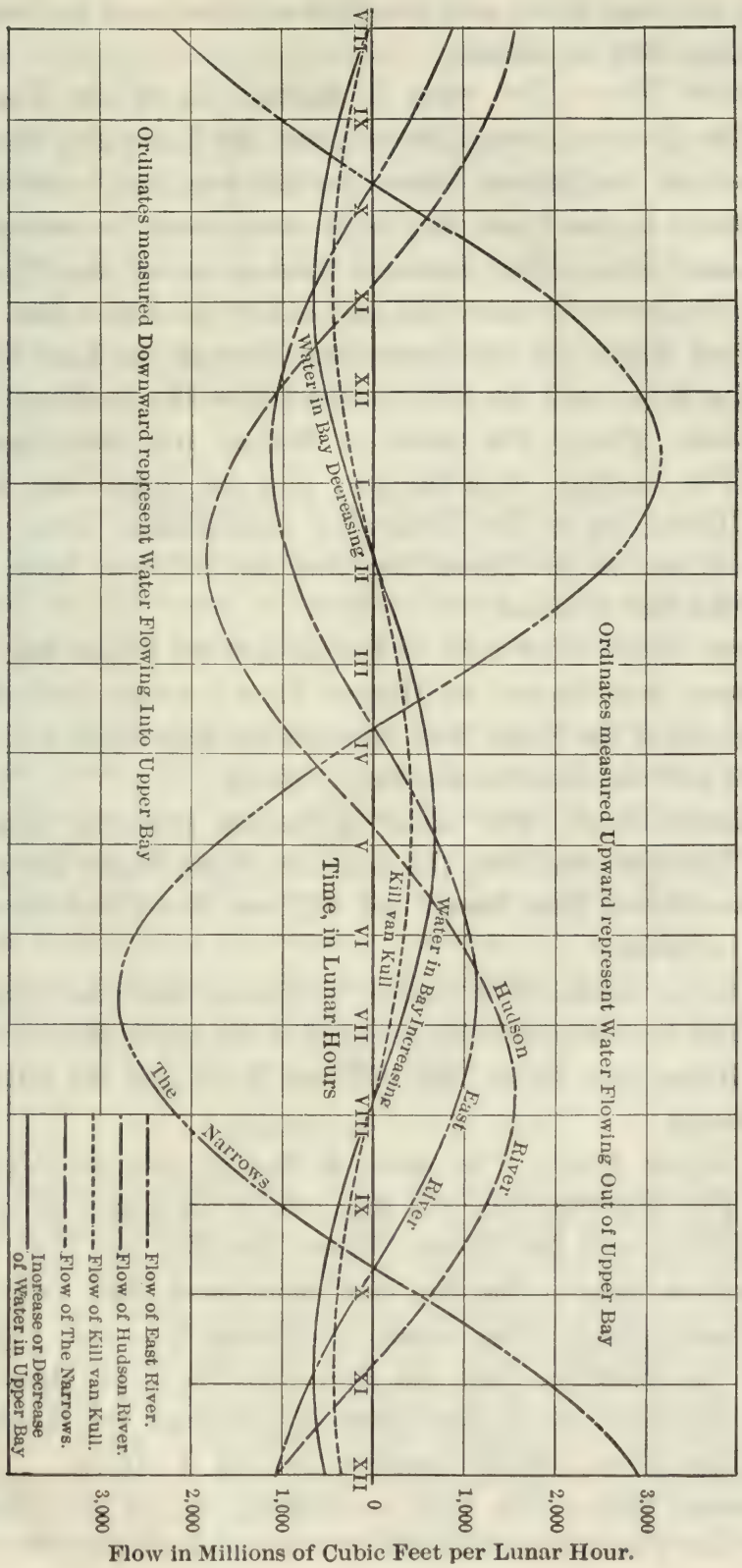

Ordinates measured Upward represent Water Flowing Out of Upper Bay
Ordinates measured Downward represent Water Flowing Into Upper Bay
Time, in Lunar Hours
VIII IX X XI XII I II III IV V VI VII VIII IX X XI XII
Water in Bay Decreasing
Water in Bay Increasing
Hudson River
East River
Kill van Kull
The Narrows
3,000 2,000 1,000 0 1,000 2,000 3,000
Flow in Millions of Cubic Feet per Lunar Hour.
Flow of East River.
Flow of Hudson River.
Flow of Kill van Kull.
Flow of The Narrows.
Increase or Decrease of Water in Upper Bay

FIG. 8.

van Kull, the East River, and the Hudson River; and the water level in the Upper Bay is falling.

II Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; into the Upper Bay through the East River and the Hudson River; the Kill van Kull is nearly slack; and the water in the Upper Bay is at about mean low water.

III Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; out of the Upper Bay, through the Kill van Kull; into the Upper Bay, through the East River and the Hudson River; and the water in the Upper Bay is rising.

IV Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; into the Upper Bay, from the Hudson River; out of the Upper Bay and Hudson River, into the East River; out of the Upper Bay, into the Kill van Kull; and the water in the Bay is rising.

V Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; the Hudson River is nearly slack; the water is flowing out of the Upper Bay, through the East River and the Kill van Kull; and the water in the Bay is rising.

VI Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; out of the Upper Bay, through the Hudson River, East River, and Kill van Kull; and the water in the Bay is rising.

VII Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; out of the Upper Bay, through the Hudson River, East River, and Kill van Kull; and the water in the Bay is rising.

VIII Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; out of the Upper Bay, through the East River and the Hudson River; the Kill van Kull is nearly slack; and the water in the Bay is at about mean high water.

IX Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; into the Upper Bay, through the Kill van Kull; out of the Upper Bay, through the Hudson River and the East River; and the water in the Bay is falling.

X Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; out of the Upper Bay, through

the Hudson River; into the Upper Bay and Hudson River, through the East River; into the Upper Bay, through the Kill van Kull; and the water in the Bay is falling.

XI Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; the Hudson River is nearly slack; the water is flowing into the Upper Bay, through the East River and the Kill van Kull; and the water in the Bay is falling.

XII Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; into the Upper Bay, through the Hudson River, East River, and Kill van Kull; and the water in the Bay is falling.

Water Levels at Each Lunar Hour.

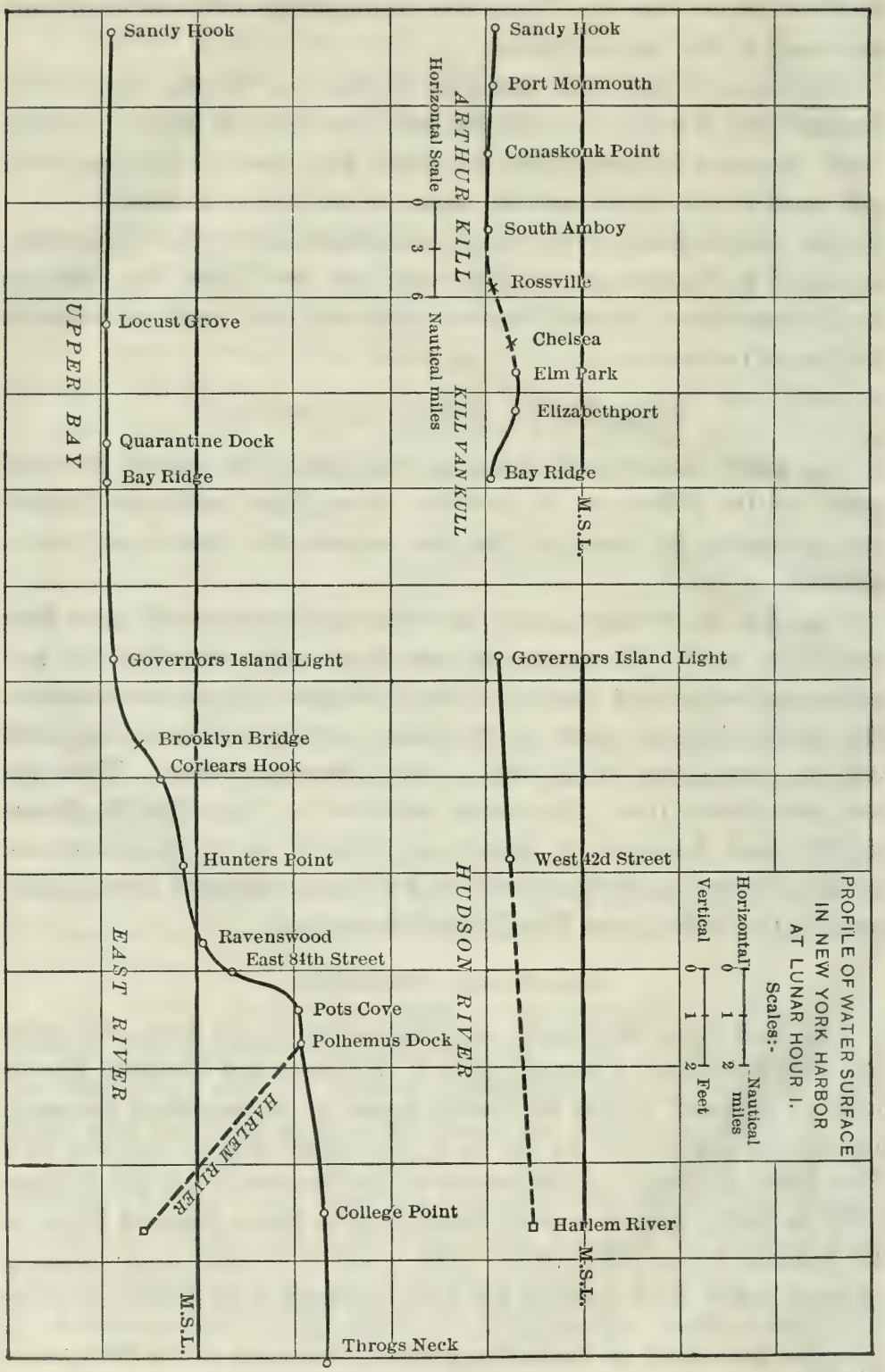
The water surfaces are changing continually throughout the tidal cycle, and the differences in elevation create slopes which are influential in causing the water to flow first in one, and then in a reverse, direction.

Figs. 9 to 20, inclusive, show the water surfaces at each lunar hour during the cycle. By comparing one figure with those for the preceding and subsequent hours, the rapid changes in levels are apparent. The figures show the cause of the violent currents through Hell Gate, and the steep slopes which exist at this interesting place. These figures were drawn from information contained in Appendix 15, Report of 1887, and Appendix 9, Report of 1888, of the U. S. Coast and Geodetic Survey, with the elevations for some additional points calculated by the writer from Tide Table information.

Land-Water Discharges.

The land-water discharged into the harbor comes from the rivers and their contributing basins. The U. S. Coast and Geodetic Survey made an estimate* of this land-water, based on the records of the gauging stations maintained by the U. S. Geological Survey and the New York State Engineer and Surveyor at Mechanicsville on the Hudson (1890 to 1905), and at a point about 4 miles below Rexford Flats on the Mohawk River (1901-1905). The quantity of land-water running off areas below these stations has been assumed to be related to these

*Letter from Survey to Metropolitan Sewerage Commission of New York, August 14th, 1908.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Governors Island Light
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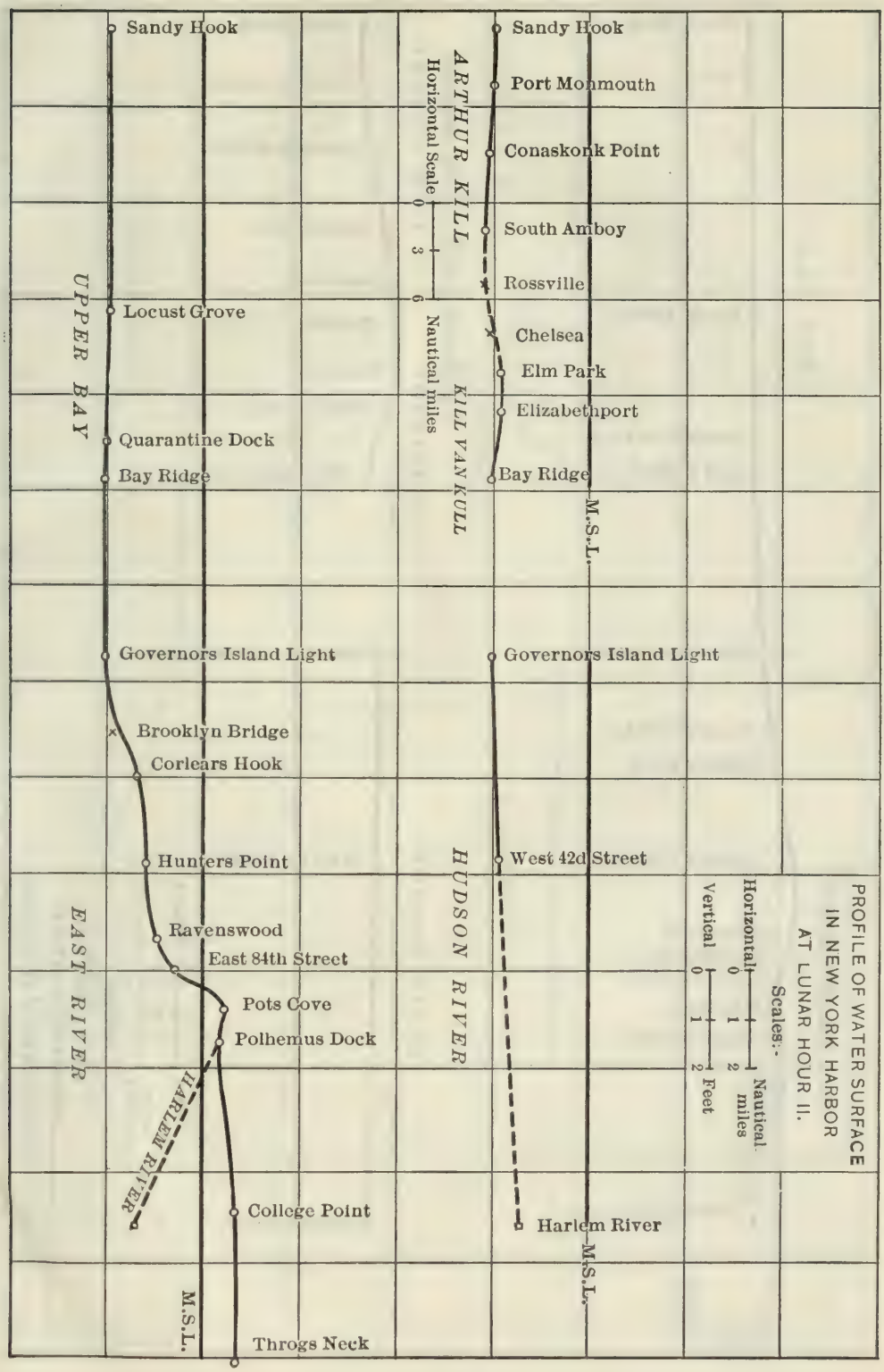
Harlem River
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
M.S.L.
ARTHUR KILL
KILL VAN KULL
UPPER BAY
HUDSON RIVER
EAST RIVER
HARLEM RIVER
Horizontal Scale
0 3 6
Nautical miles
PROFILE OF WATER SURFACE IN NEW YORK HARBOR AT LUNAR HOUR 1.
Scales:-
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet

FIG. 9.


PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR 11.
Scales:-
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
Horizontal Scale 0 3 6 Nautical miles
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
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Elm Park
Elizabethport
Bay Ridge
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Quarantine Dock
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Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
Governors Island Light
West 42d Street
Harlem River
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
HARLEM RIVER
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EAST RIVER
M.S.L.
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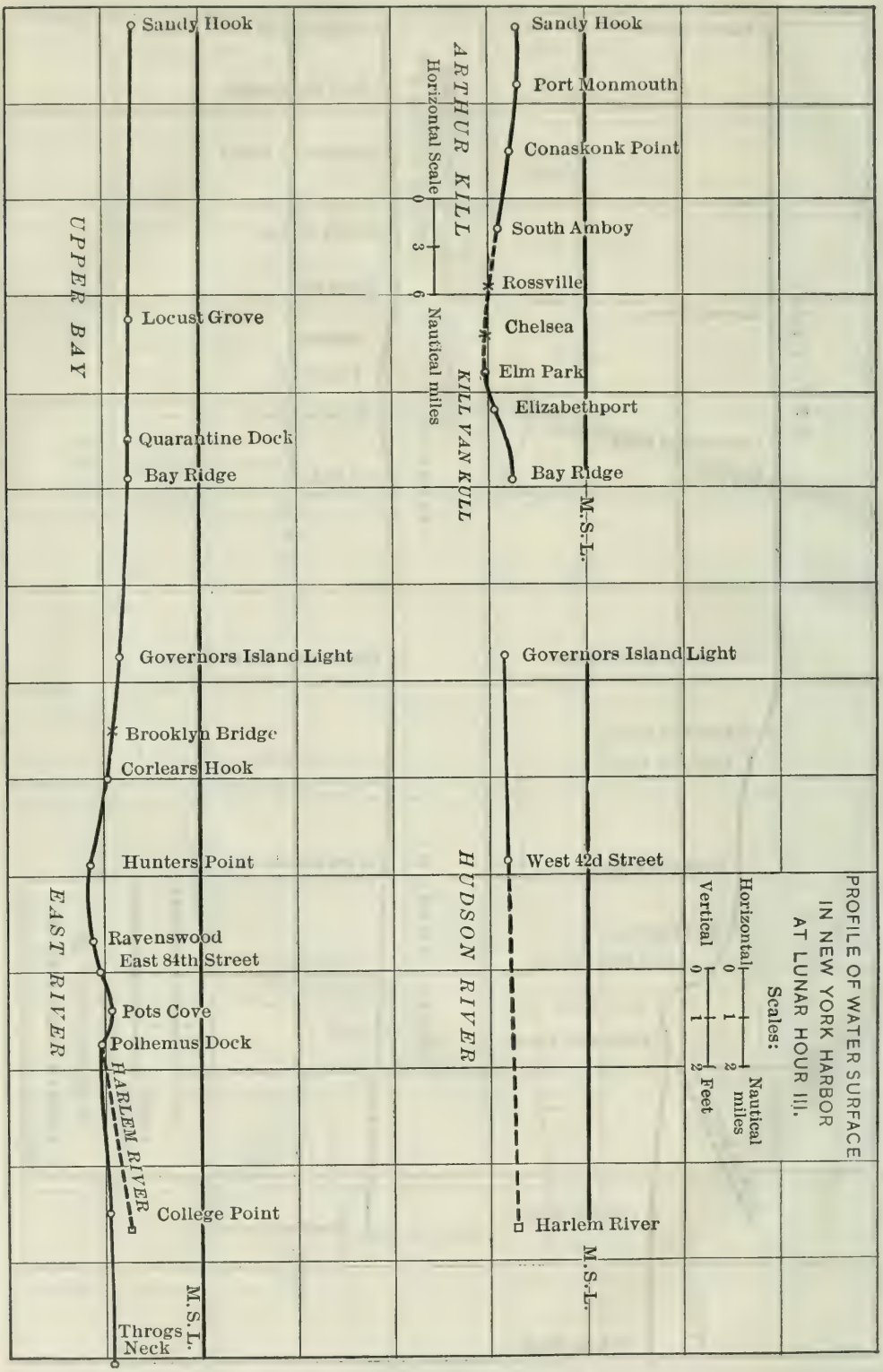
M.S.L.

FIG. 10.


PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR 11.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
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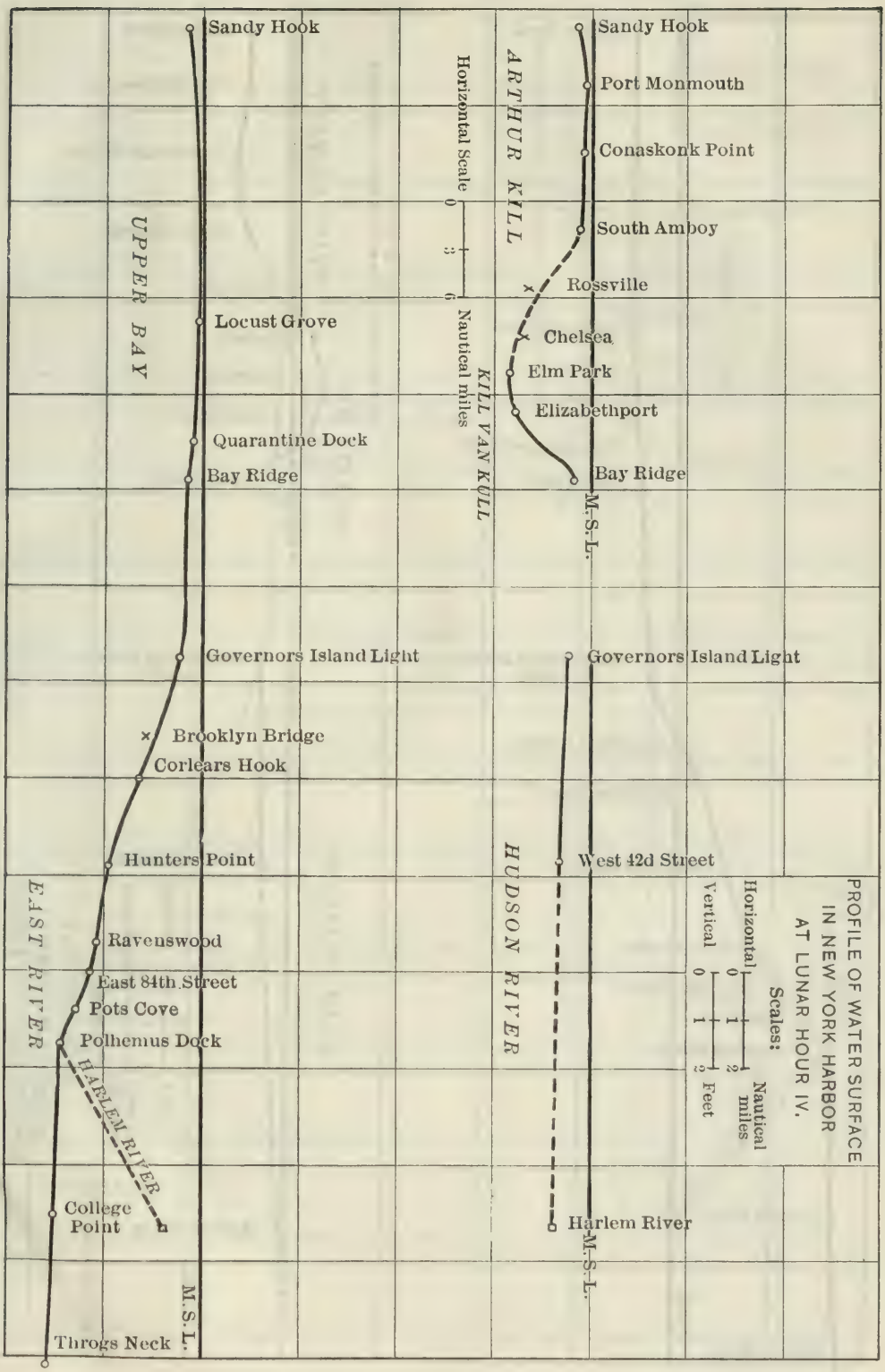
Elm Park
Elizabethport
Bay Ridge
M.S.L.
Governors Island Light
West 42d Street
Harlem River
M.S.L.
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
Horizontal Scale 0 3 6 Nautical miles
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
M.S.L.
Throgs Neck
UPPER BAY
EAST RIVER
HARLEM RIVER

Fig. 11.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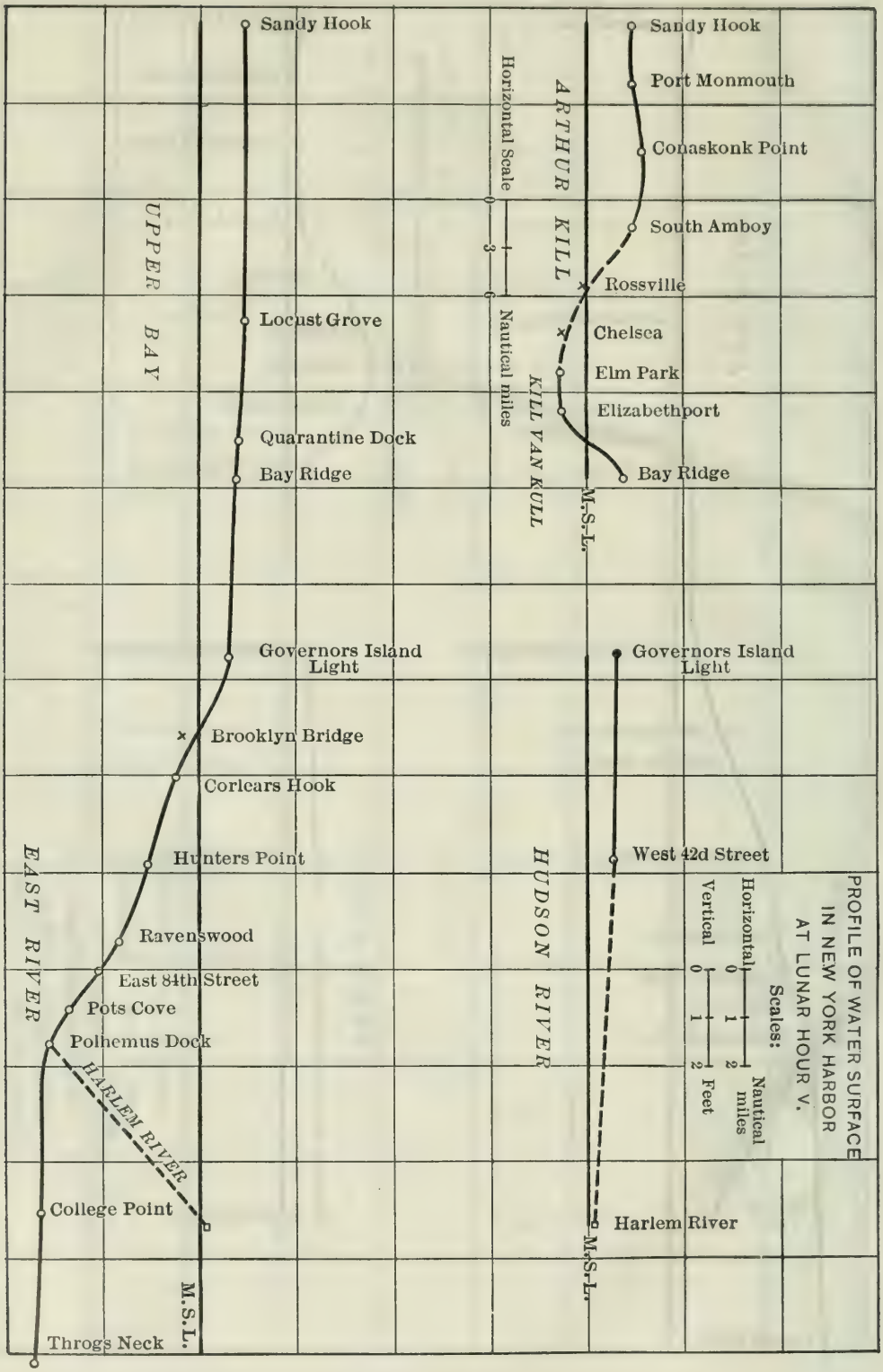
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
UPPER BAY
KILL VAN KULL
Horizontal Scale
0 3 6
Nautical miles
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
EAST RIVER
HARLEM RIVER
College Point
Throgs Neck
M.S.L.
Governors Island Light
West 42d Street
HUDSON RIVER
Harlem River
M.S.L.
PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR IV.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet

Fig. 12.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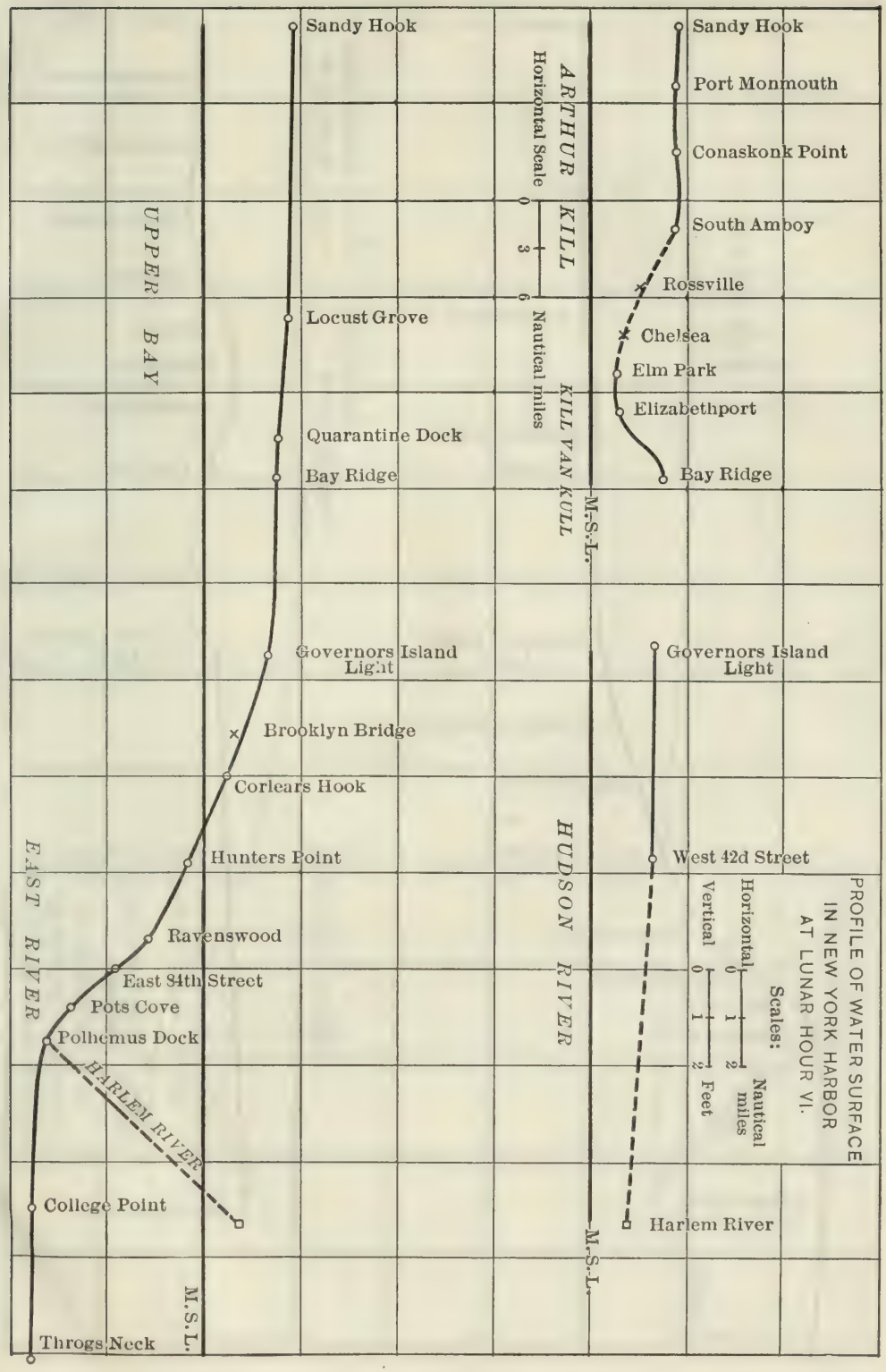
West 42d Street
Harlem River
ARTHUR KILL
KILL VAN KULL
UPPER BAY
EAST RIVER
HUDSON RIVER
HARLEM RIVER
M.S.L.
Horizontal Scale
Nautical miles
PROFILE OF WATER SURFACE IN NEW YORK HARBOR AT LUNAR HOUR V.
Scales:
Horizontal
Vertical
Nautical miles
Feet

FIG. 13.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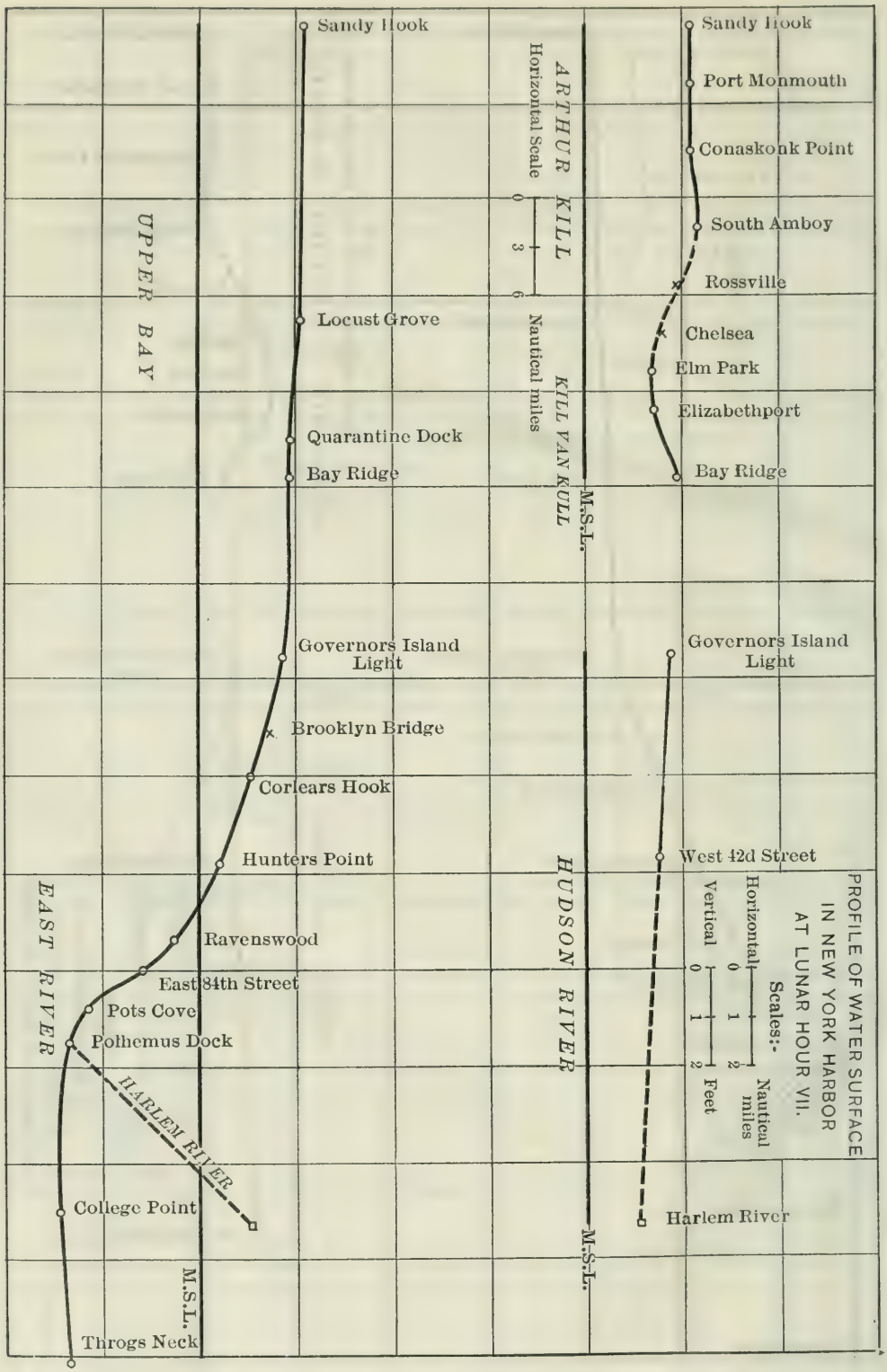
College Point
Throgs Neck
ARTHUR KILL
KILL VAN KULL
UPPER BAY
EAST RIVER
HUDSON RIVER
M.S.L.
M.S.L.
M.S.L.
Horizontal Scale
0
3
6
Nautical miles
Governors Island Light
West 42d Street
Harlem River
PROFILE OF WATER SURFACE IN NEW YORK HARBOR AT LUNAR HOUR VI.
Scales:
Horizontal
0 1 2
Nautical miles
Vertical
0 1 2
Feet

FIG. 14.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Governors Island Light
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Harlem River
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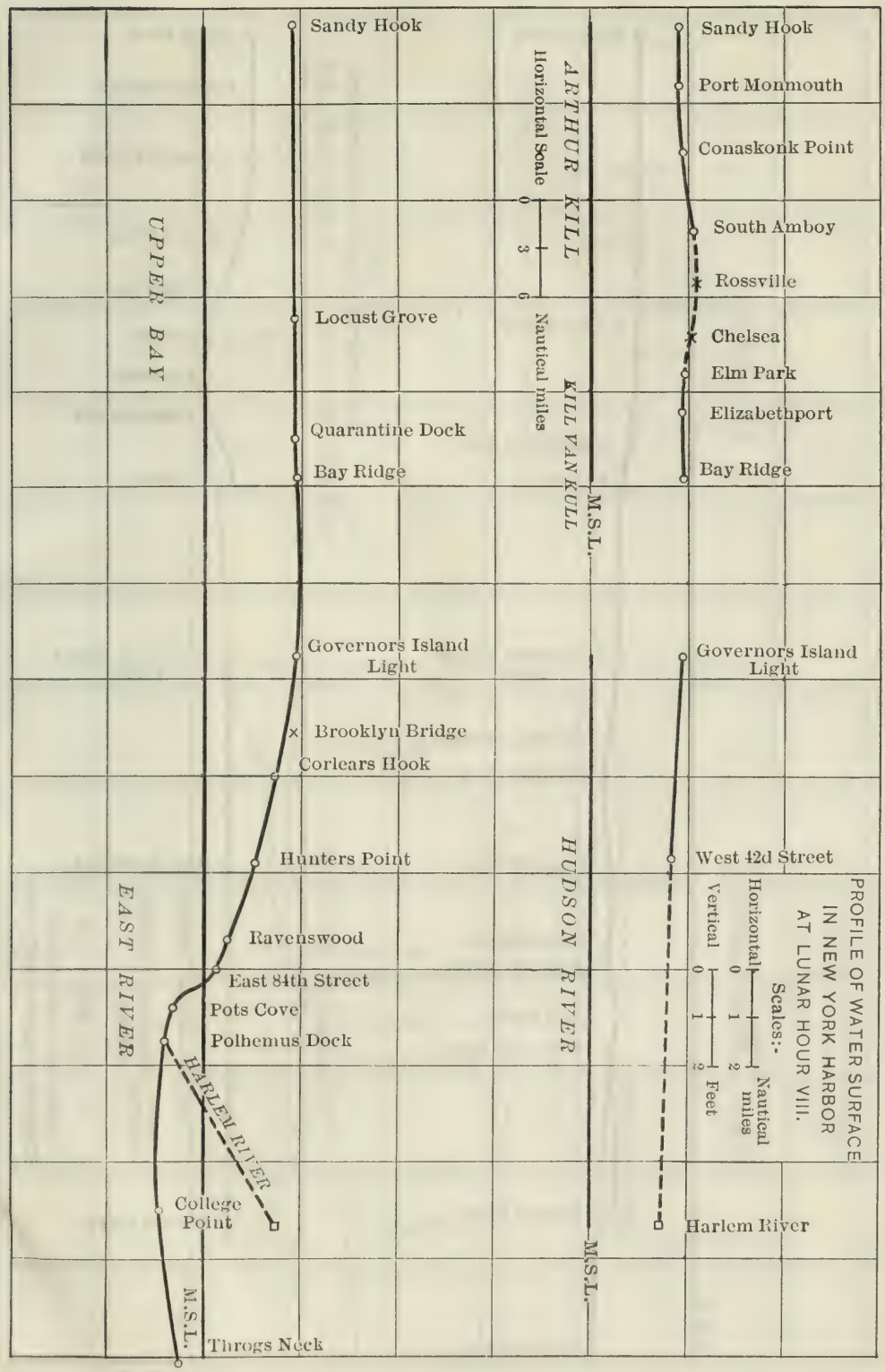
UPPER BAY
EAST RIVER
HUDSON RIVER
HARLEM RIVER
M.S.L.
M.S.L.
M.S.L.
Horizontal Scale
0
3
6
Nautical miles
PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR VII.
Scales:-
Horizontal
0
1
2
Nautical miles
Vertical
0
1
2
Feet

Fig. 15.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ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
Horizontal Scale
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Nautical miles
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Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
HARLEM RIVER
College Point
Throgs Neck
M.S.L.
UPPER BAY
EAST RIVER
PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR VIII.
Scales:-
Horizontal
0 1 2
Nautical miles
Vertical
0 1 2
Feet

Fig. 16.

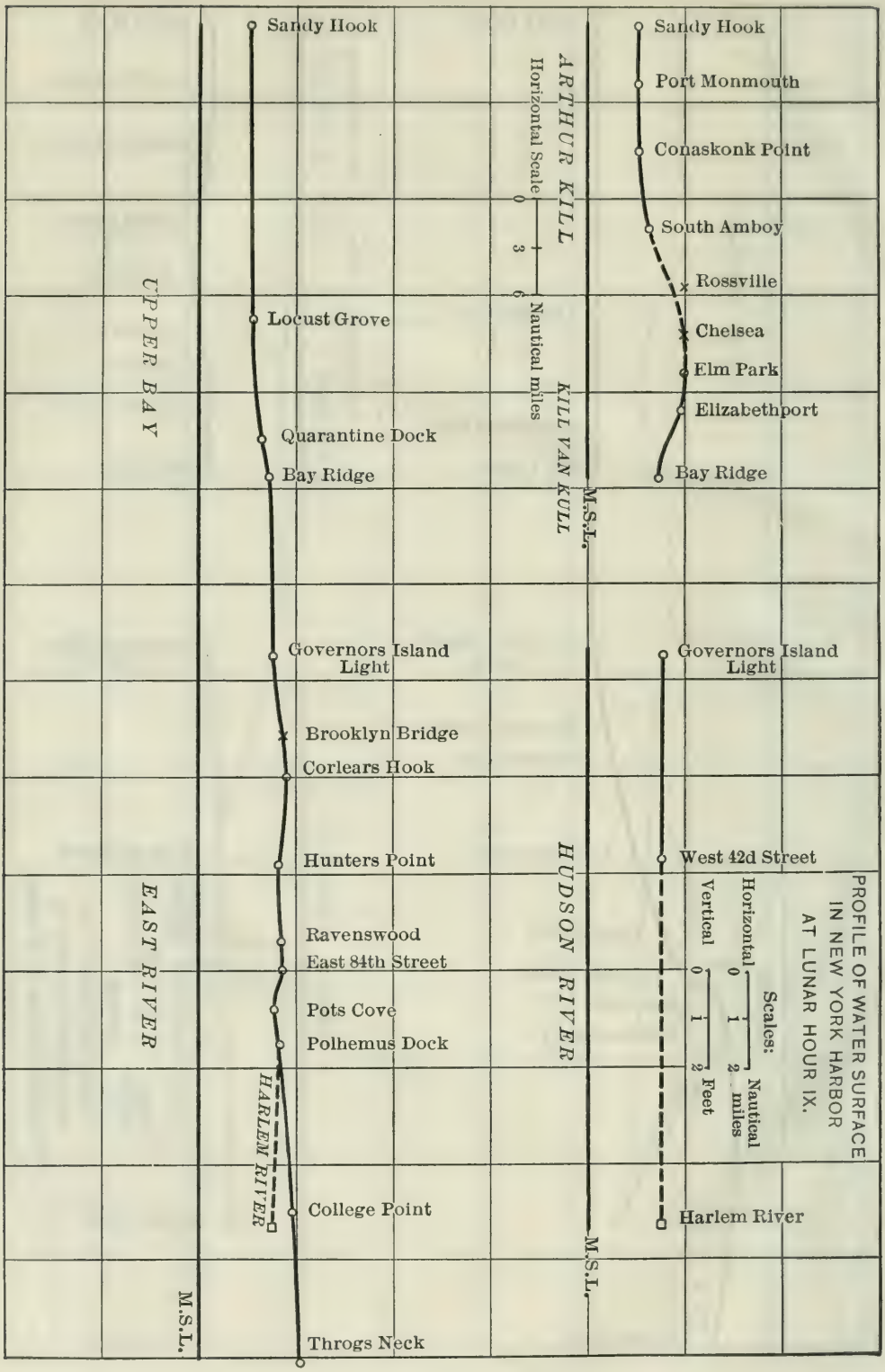
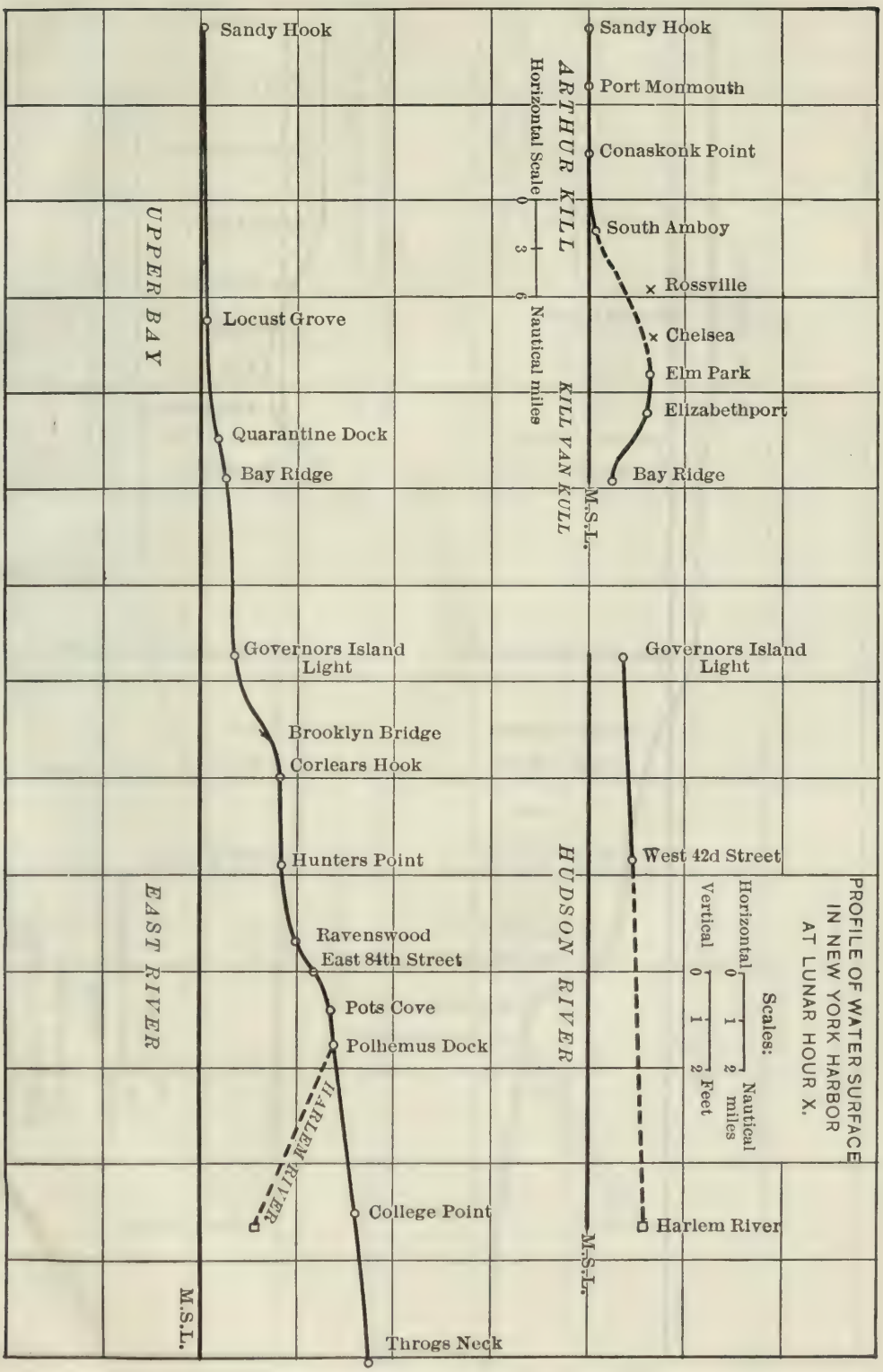

PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR IX.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
Governors Island Light
West 42d Street
Harlem River
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
M.S.L.
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
EAST RIVER
UPPER BAY
HARLEM RIVER
Horizontal Scale
0 3 6
Nautical miles

Fig. 17.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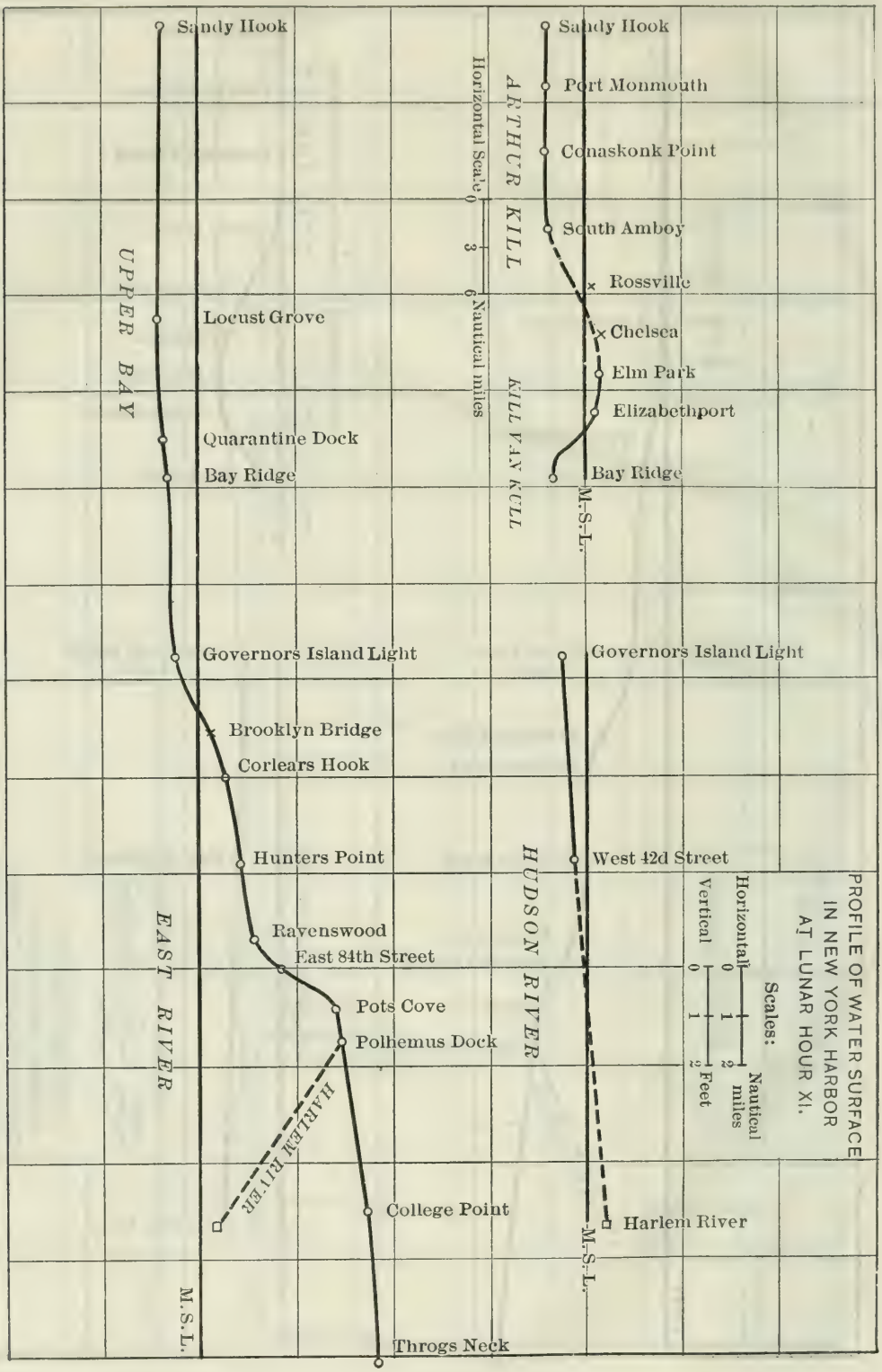
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
HARLEM RIVER
College Point
Throgs Neck
M.S.L.
Governors Island Light
West 42d Street
Harlem River
M.S.L.
ARTHUR KILL
KILL VAN KULL
UPPER BAY
EAST RIVER
HUDSON RIVER
Horizontal Scale
0
3
6
Nautical miles
PROFILE OF WATER SURFACE IN NEW YORK HARBOR AT LUNAR HOUR X.
Scales:
Horizontal
0 1 2 Nautical miles
Vertical
0 1 2 Feet

FIG. 18.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Throgs Neck
Governors Island Light
West 42d Street
Harlem River
ARTHUR KILL
KILL VAN KULL
UPPER BAY
EAST RIVER
HUDSON RIVER
HARLEM RIVER
M.S.L.
M.S.L.
M.S.L.
Horizontal Scale
0 3 6 Nautical miles
PROFILE OF WATER SURFACE IN NEW YORK HARBOR AT LUNAR HOUR XI.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet

Fig. 19.

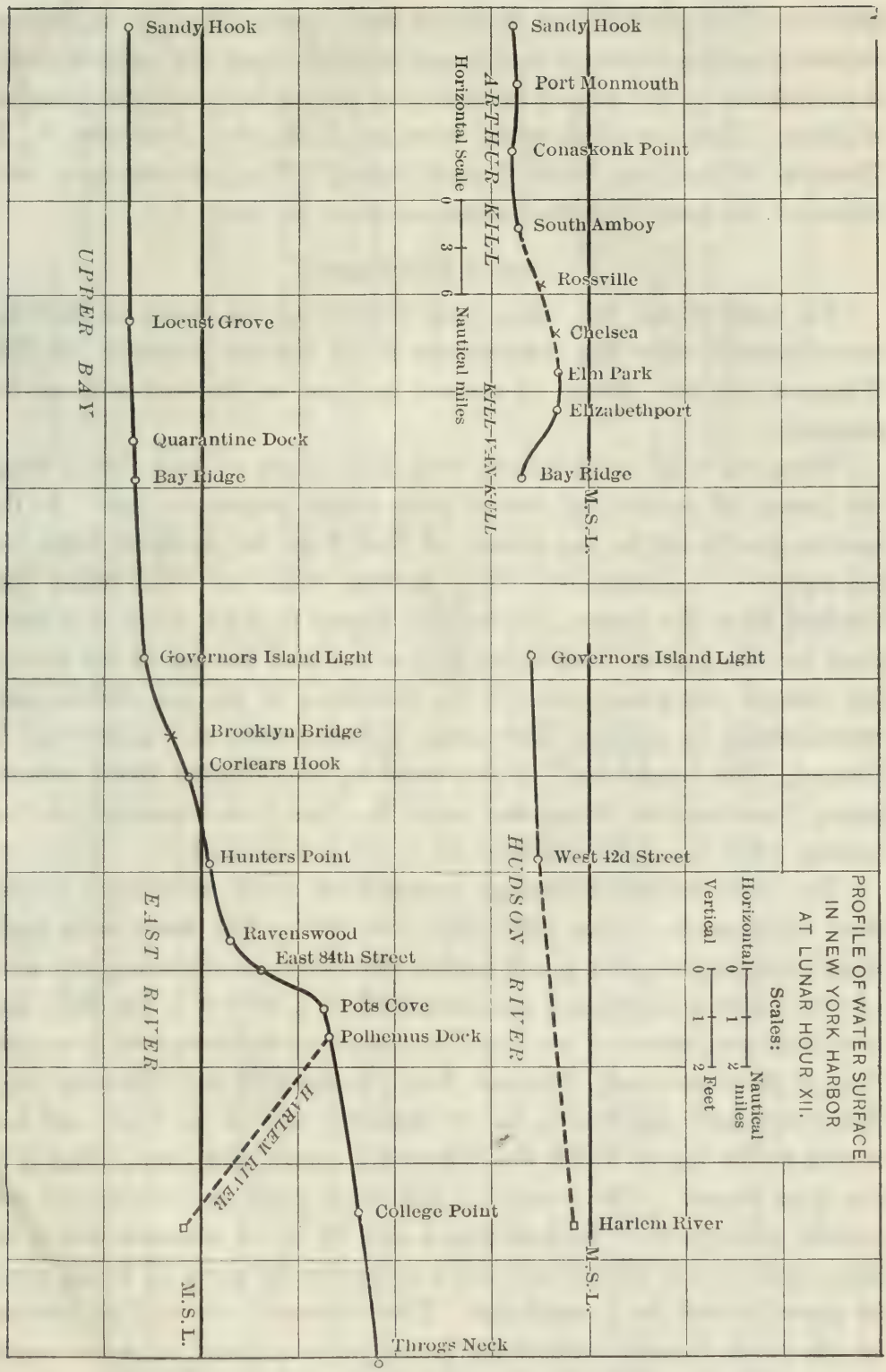

PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR XII.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
M.S.L.
ARTHUR KILL
KILL VAN KULL
Horizontal Scale 0 3 6 Nautical miles
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
UPPER BAY
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
EAST RIVER
HARLEM RIVER
College Point
M.S.L.
Throgs Neck
Governors Island Light
West 42d Street
HUDSON RIVER
Harlem River
M.S.L.

Fig. 20.

areas as the measured discharges are to the areas above the gauging stations. The results thus obtained were then multiplied by factors representing the ratios of the annual rainfalls over the regions under consideration to the annual rainfall over the regions above the gauging stations. These rainfall ratios were obtained from Professor A. J. Henry's "Climatology of the United States." The final results or estimates of the land-water discharges are given in Table 5.

Current Phenomena.

The turn of the tide takes place first along the shore, so that the shore currents often flow contrariwise to the channel currents. In The Narrows, the first change of current appears on the east side of the channel.

When the tidal currents turn from flood to ebb or from ebb to flood, the inertia of the moving stream plays a very important part. As the specific gravity of the sea outside of New York bar is about 1.024, the sea-water is approximately 2½% heavier than the fresh-water discharged from the rivers. As the tide begins to flood, there is a tendency for this lighter fresh-water to flow out over the top of the incoming heavier salt-water, and for the formation of top and bottom currents flowing in opposite directions. Consequently, an "under-run" is formed. The "under-run" of salt-water up the Hudson River extends above Poughkeepsie, where the water is often found brackish during seasons when the river discharge of land-water is small.

The Metropolitan Sewerage Commission made more than ninety float experiments during 1907, 1908, and 1909. The floats were made deep, in order that they might not be influenced by the wind, and were of two kinds: a submerged can supported by a wire to a can float, and a wooden post carrying vanes at the lower end supported by a wooden float at the upper end. The can float was made in the following way: The submerged can was 6¼ in. in diameter and 14 in. high, and had a ring at the top to which was fastened a copper wire connecting it to the float proper. The length of this wire could be varied, but was usually about 4 ft. The float was a can, 5⅜ in. in diameter and 5 in. high, hermetically sealed, and had a socket on the cover in which could be placed a staff for a small flag. The submerged can had an opening in the cover through which sand could be introduced so as to weight the apparatus until the float became almost submerged.

TABLE 5.—AVERAGE DISCHARGE OF FRESH WATER, IN CUBIC FEET PER SIX LUNAR HOURS, IN NEW YORK HARBOR.

Contributing basin.	Area, in square miles.	Jan.	Feb.	Mar.	Apr.	May.	June.
Hudson River (above Battery Place)	13 369	405 467 000	317 536 000	1 149 373 000	1 190 980 000	615 645 000	500 014 000
East River, between Governors Island and Randalls Island	36	1 274 000	984 000	3 577 000	3 689 000	1 900 000	1 565 000
Newark Bay and Kill van Kull, ignoring discharge through Arthur Kill	1 139	39 348 000	30 808 000	111 561 000	115 608 000	59 738 000	48 515 000
Newark Bay and Kill van Kull, allowing 83.66313% for the proportion from Newark Bay flowing through the Kill van Kull	953	32 920 000	25 775 000	93 335 000	96 721 000	49 979 000	40 589 000
Upper Bay (exclusive of all above tributaries)	37	1 297 000	1 028 000	3 689 000	3 823 000	1 990 000	1 610 000
Total entering Upper Bay above The Narrows	14 395	440 958 000	345 323 000	1 249 974 000	1 295 213 000	669 514 000	543 778 000
Arthur Kill, allowing 16.33687% for proportion from Newark Bay	353	12 330 000	9 661 000	34 949 000	36 214 000	18 724 000	15 214 000
Raritan River	1 105	37 314 000	29 221 000	105 771 000	109 594 000	56 653 000	46 011 000
*Total entering Lower Bay, exclusive of that entering Upper Bay	1 458	49 644 000	38 882 000	140 720 000	145 808 000	75 377 000	61 225 000

Contributing basin.	Area, in square miles.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
Hudson River (above Battery Place)	13 369	320 734 000	286 371 000	325 652 000	453 042 000	419 216 000	539 027 000	543 588 000
East River, between Governors Island and Randalls Island	36	984 000	894 000	1 028 000	1 408 000	1 297 000	1 677 000	1 690 000
Newark Bay and Kill van Kull, ignoring discharge through Arthur Kill	1 139	31 143 000	27 790 000	31 613 000	43 976 000	40 667 000	52 315 000	52 757 000
Newark Bay and Kill van Kull, allowing 83.66313% for the proportion from Newark Bay flowing through the Kill van Kull	953	26 055 000	23 250 000	26 448 000	36 792 000	34 023 000	43 768 000	44 138 000
Upper Bay (exclusive of all above tributaries)	37	1 028 000	917 000	1 051 000	1 476 000	1 364 000	1 744 000	1 751 000
Total entering Upper Bay above The Narrows	14 395	348 801 000	311 432 000	354 179 000	492 718 000	455 900 000	586 216 000	591 167 000
Arthur Kill allowing 16.33687% for proportion from Newark Bay	353	9 761 000	8 698 000	9 905 000	13 757 000	12 725 000	16 394 000	16 528 000
Raritan River	1 105	29 511 000	26 337 000	29 958 000	41 673 000	38 588 000	49 610 000	50 002 000
*Total entering Lower Bay, exclusive of that entering Upper Bay	1 458	39 272 000	35 035 000	39 863 000	55 430 000	51 313 000	66 004 000	66 530 000

NOTE.—Of the water entering and leaving Newark Bay and tributaries, 83.66313% passes Kill van Kull and 16.33687% passes the upper end of Arthur Kill.

Annual rainfall:
- Hudson and Mohawk Valleys, above gauging stations ..... 37 in.
- Hudson Valley below gauging stations ..... 41 "
- East River ..... 44½ "
- Newark Bay Basin ..... 44 in.
- Upper Bay ..... 45 "
- Arthur Kill Basin ..... 45 "
- Raritan River Basin ..... 43 "

*Total from Arthur Kill, Newark Bay, and Raritan River, but not Shrewsbury River or similar drainage areas.

The second float consisted of a 4 by 4-in. wooden standard, varying in length, but usually about 6 ft. long. At the lower end, four 18 by 18-in. vanes of No. 14 B. w. g. sheet-iron were bolted. At the upper end a 12 by 12 by 24-in. wooden block was fastened vertically, and the whole was weighted so that it was nearly submerged. On the upper end of the float there were iron straps which carried two heavy hooks for fastening the lifting tackle, and a light rod on which could be put a flag during the day or a lantern at night. The second type of float gave the greater satisfaction, as it was the more substantial for this heavy service, and its motion was not affected appreciably by the wind.

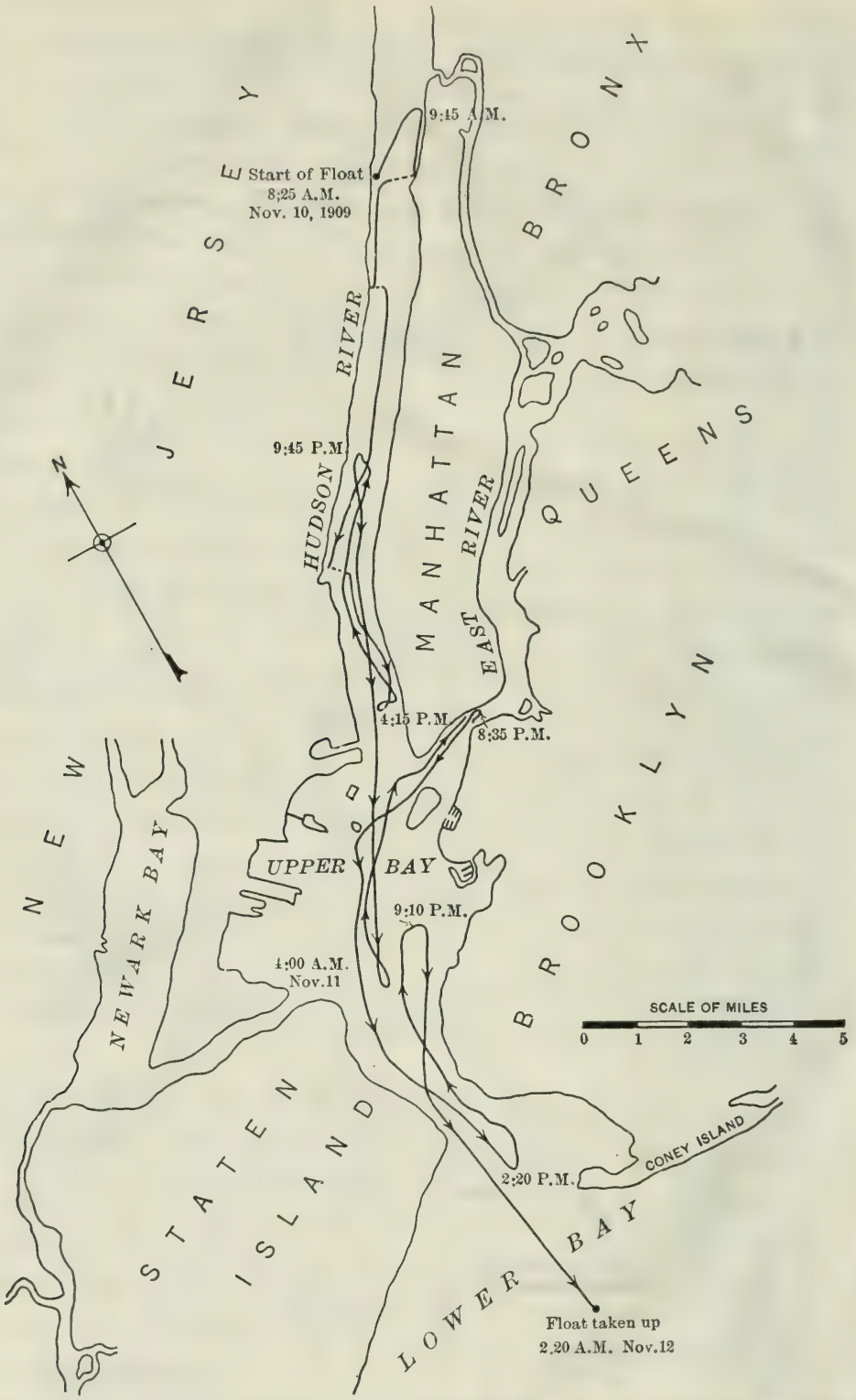
Floats were set adrift in different parts of the harbor and at different stages of the tide. They were followed by observers who noted their position at frequent intervals, generally using the sextant, and the positions were plotted on a large-scale chart. Many of the paths thus traced were duplicated by other floats. Broadly speaking, the floats showed a general trend toward the sea, past Sandy Hook; although, naturally, they were carried forward and backward with the ebb and flood tides. Figs. 21 to 32, inclusive, show the paths of twelve representative floats, as observed by the Metropolitan Sewerage Commission. Fig. 33 shows the paths of floats at the entrance to Jamaica Bay as observed under the direction of Col. John G. D. Knight, Corps of Engineers, U. S. Army, in December, 1908. Some were started in the ocean and entered the bay on flood tide, others were set adrift in the entrance and passed out on ebb tide. The tendency is for the currents to flow into and out of the bay around Rockaway Point. The flow is chiefly to and from the ocean, and less directly past Coney Island.

The paths of the floats show the general behavior of the currents in the harbor; the general drift toward the Atlantic Ocean; the backward and forward paths in the Upper Bay and rivers; the tendency to hug the Staten Island shore at The Narrows; the tendency of a particle to remain in Newark Bay and Kill van Kull; and the tendency of a particle to remain in the East River (Figs. 27, 28, 29, and 30); which substantiates the conclusion that the southerly resultant flow of the East River is not great.

When velocity data are worked out from float experiments, a correction should be made for the range of tide existing on the day of

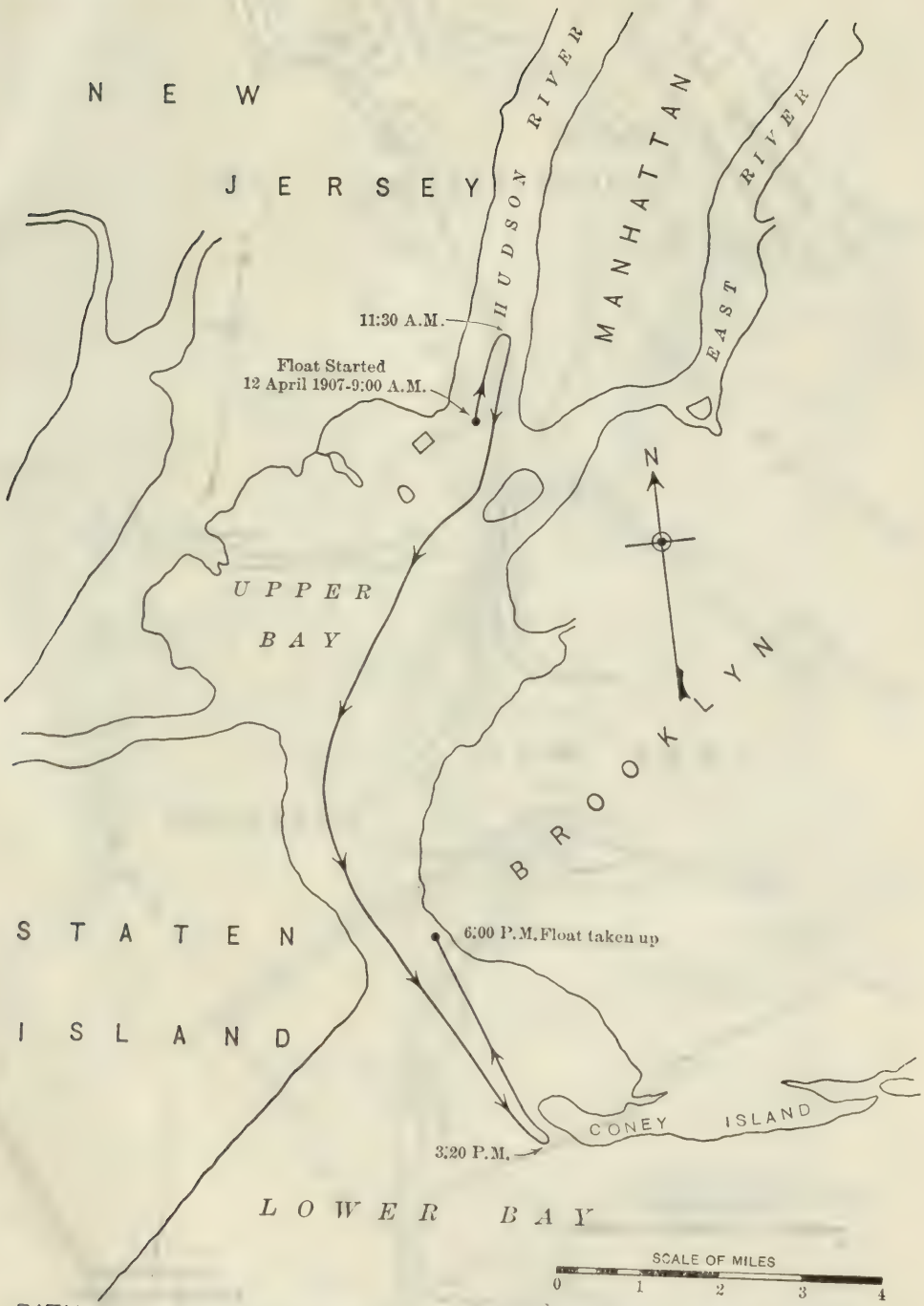
PATH OF A FLOAT FROM NEWARK BAY TO THE SEA.

FIG. 21.



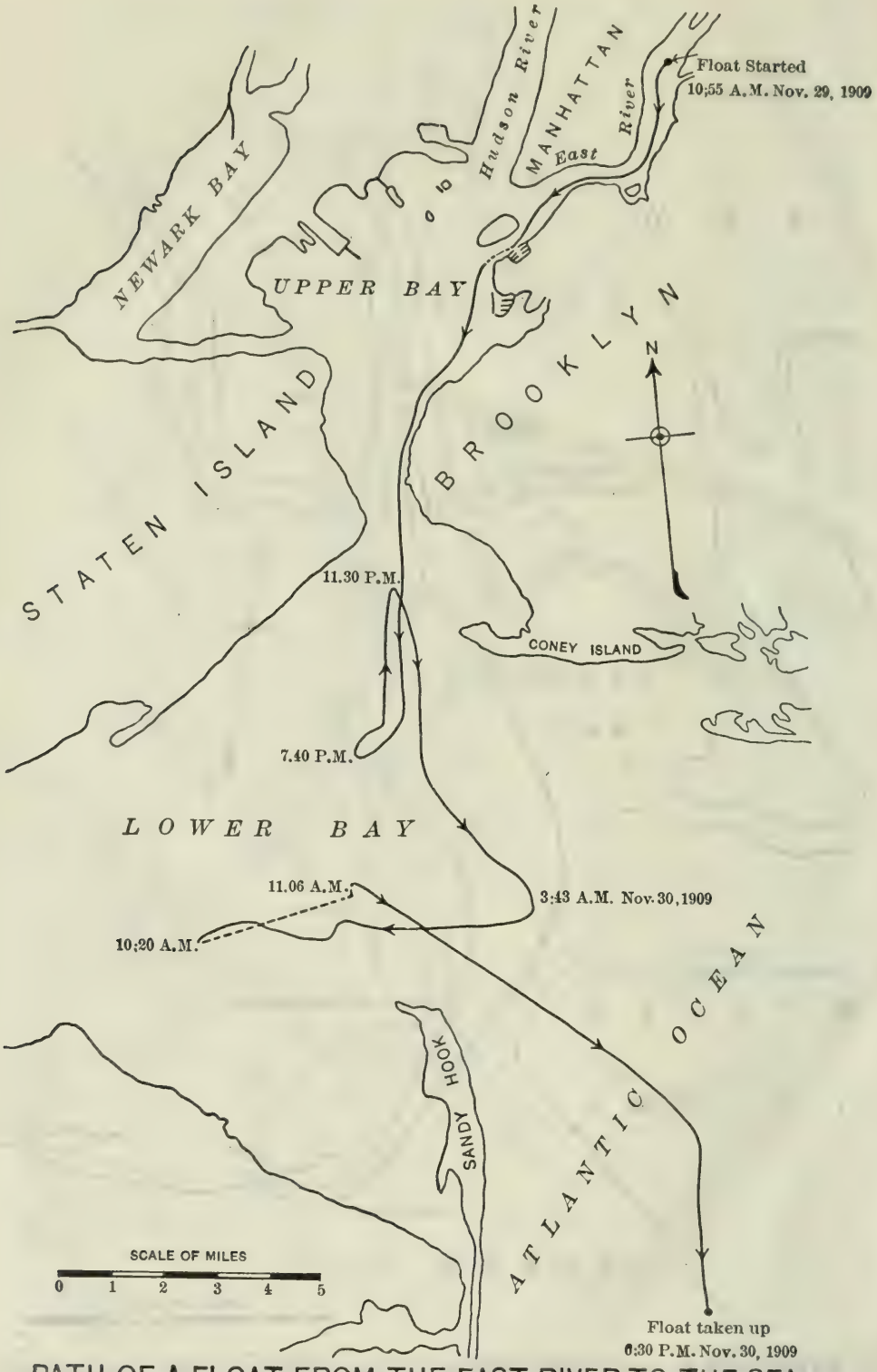
PATH OF A FLOAT FROM THE HUDSON RIVER TO THE LOWER BA

FIG. 22.



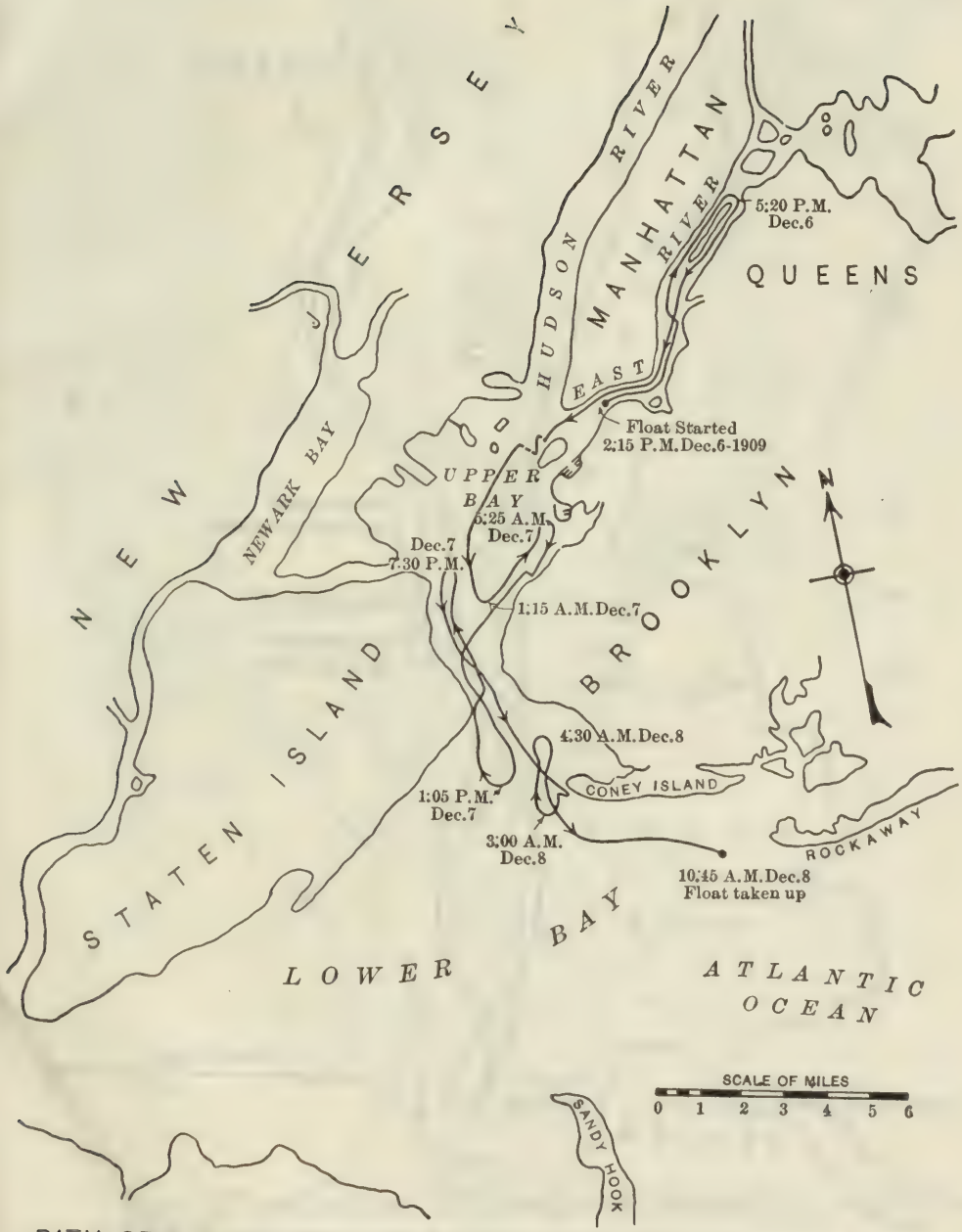
PATH OF A FLOAT FROM THE HUDSON RIVER TO THE LOWER BAY

FIG. 23.



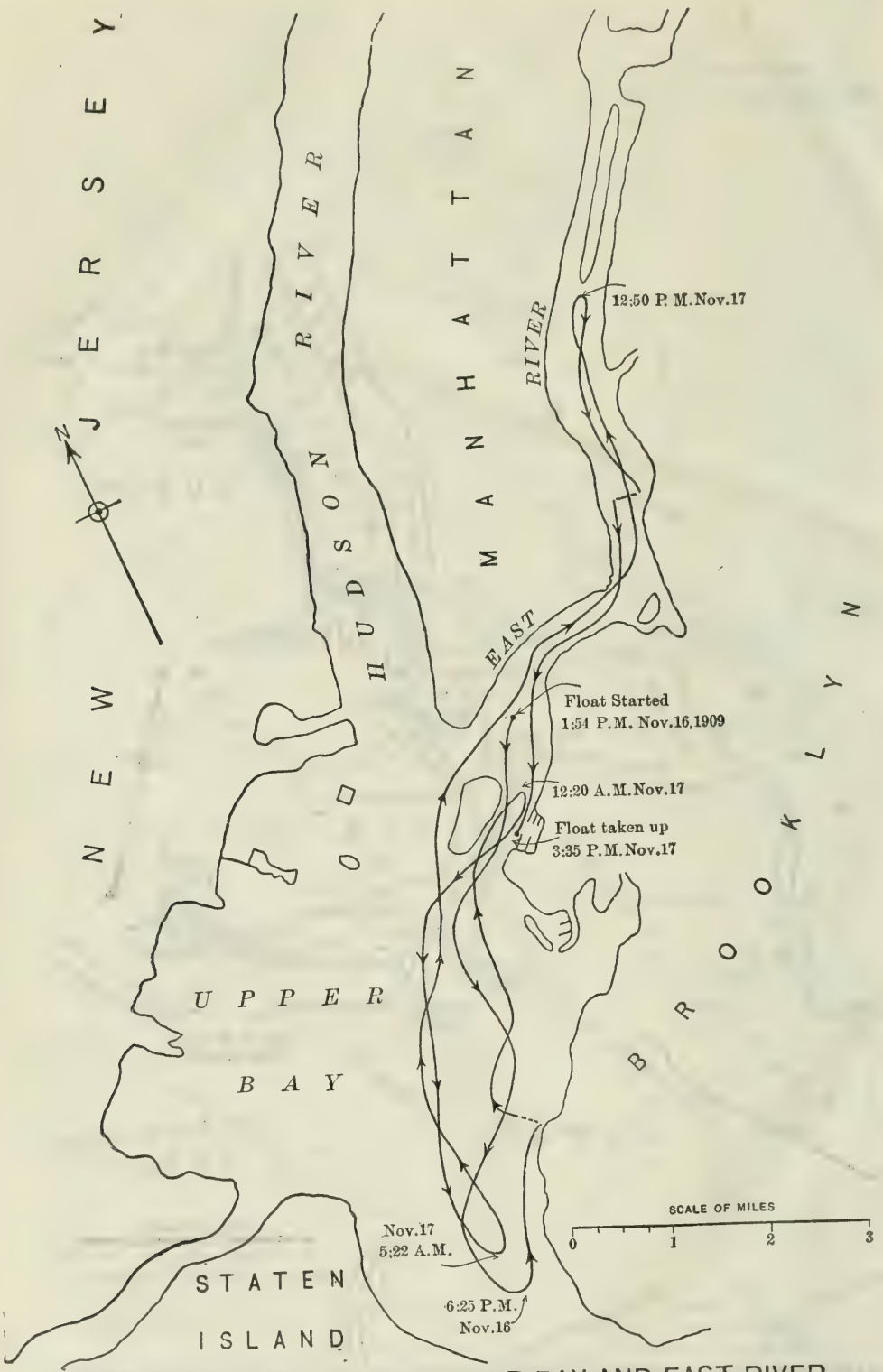
PATH OF A FLOAT FROM THE EAST RIVER TO THE SEA

FIG. 24.



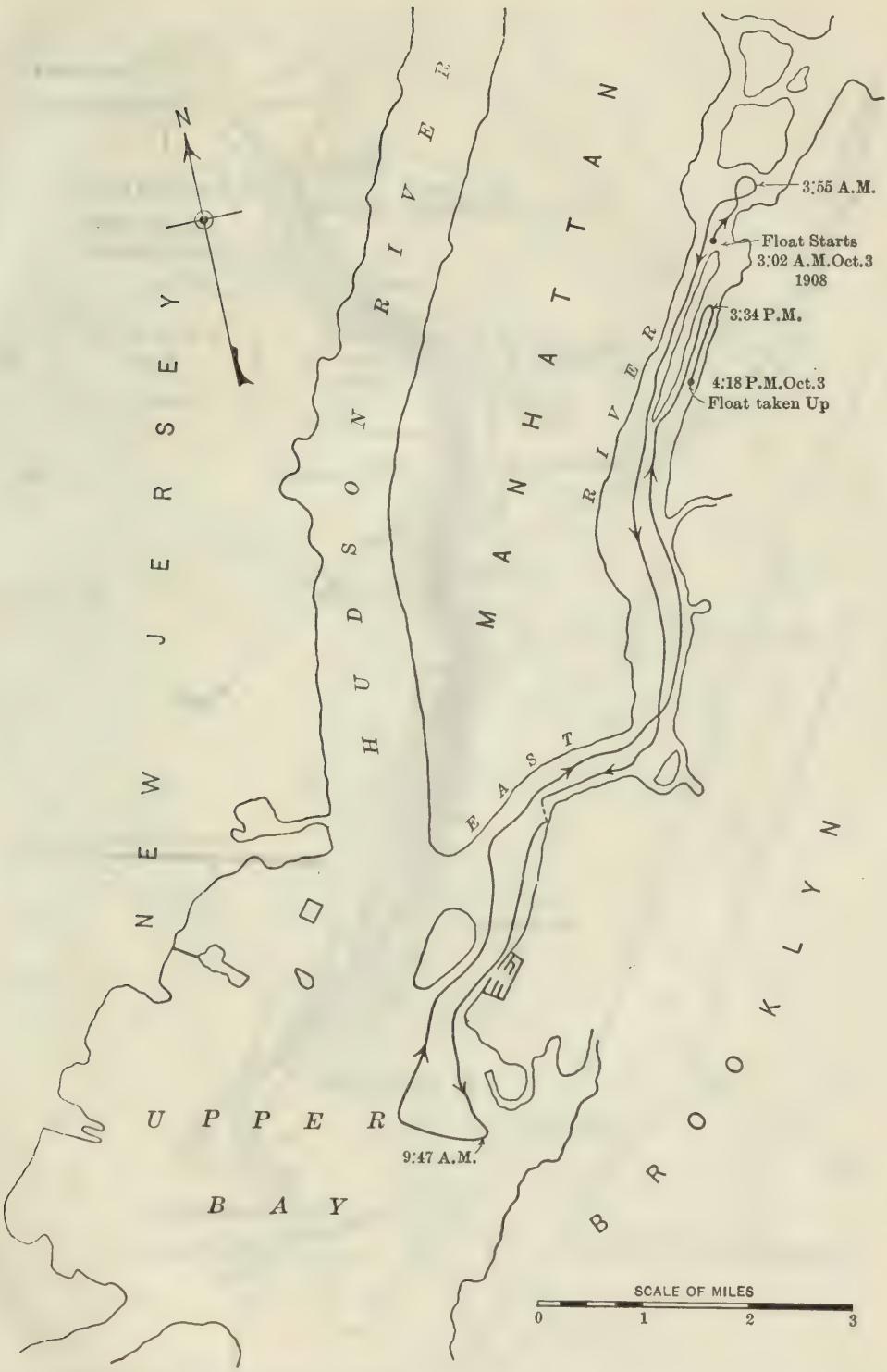
PATH OF A FLOAT FROM THE EAST RIVER TO THE LOWER BAY

FIG. 25.



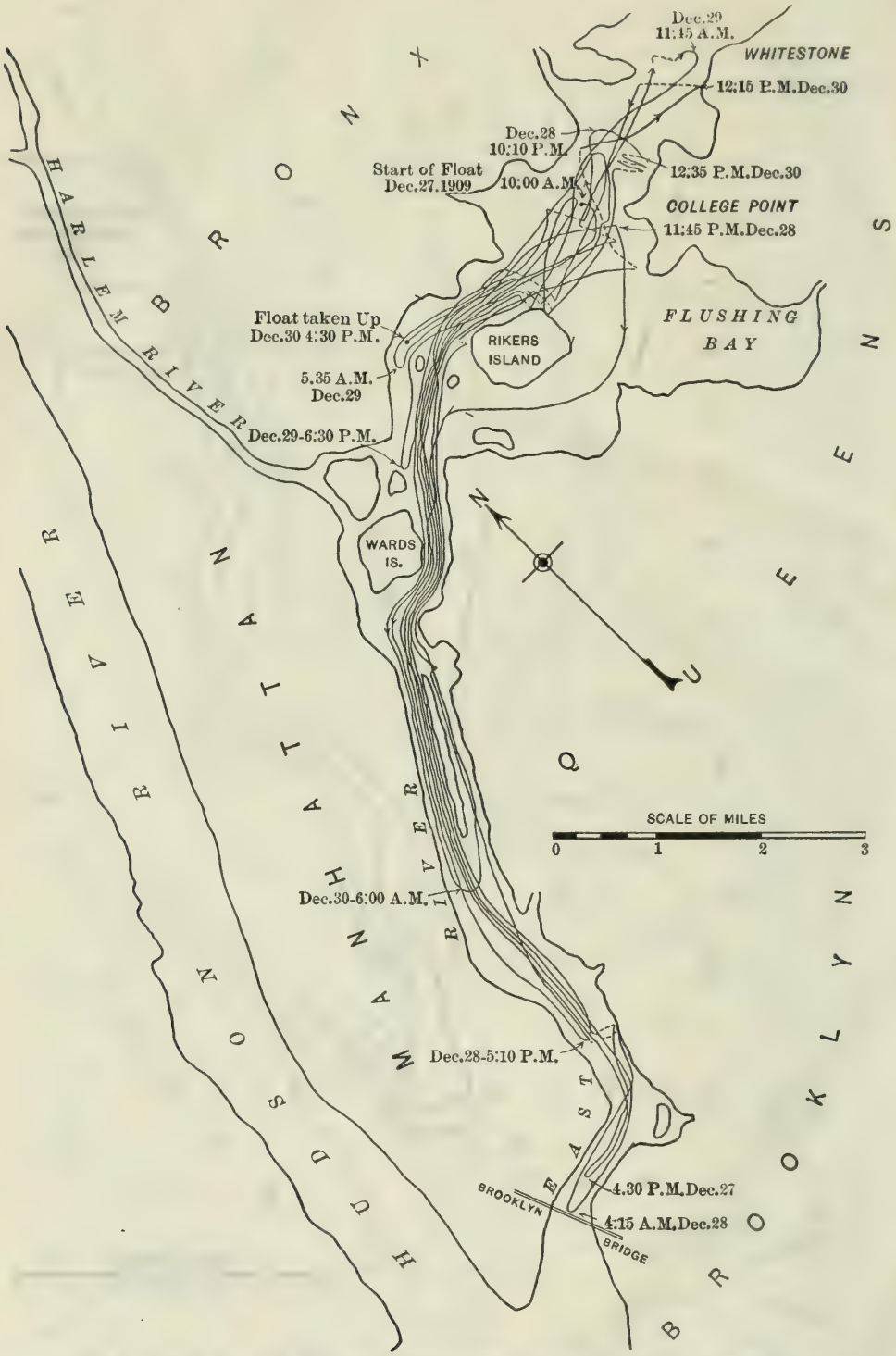
PATH OF A FLOAT IN THE UPPER BAY AND EAST RIVER

FIG. 26.



PATH OF A FLOAT IN THE UPPER BAY TO THE EAST RIVER

FIG. 27.



PATH OF A FLOAT IN THE EAST RIVER

FIG. 28.

PATH OF A FLOAT IN THE EAST RIVER.

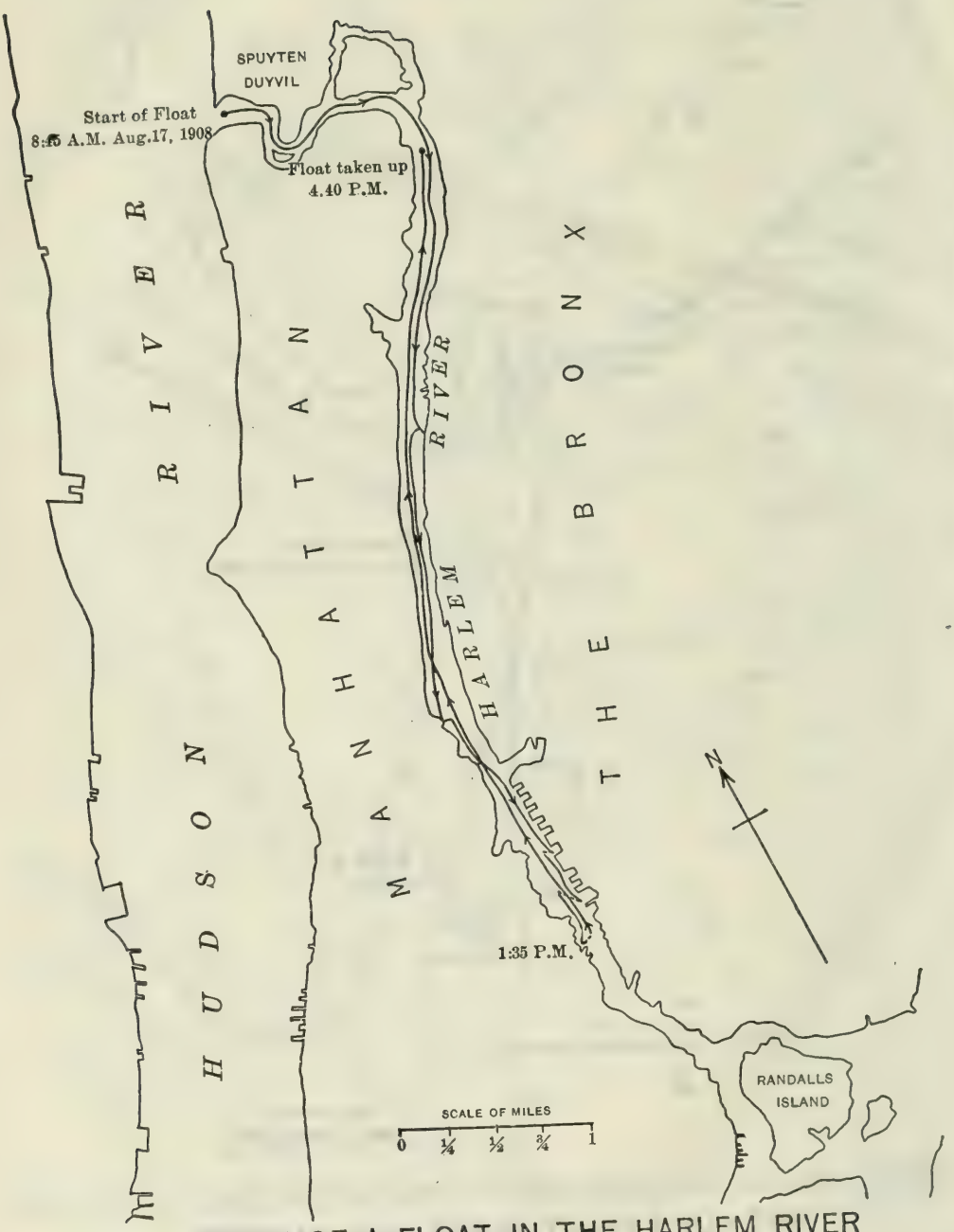
FIG. 29.

PATH OF A FLOAT IN THE EAST RIVER AND LONG ISLAND SOUND

FIG. 30.

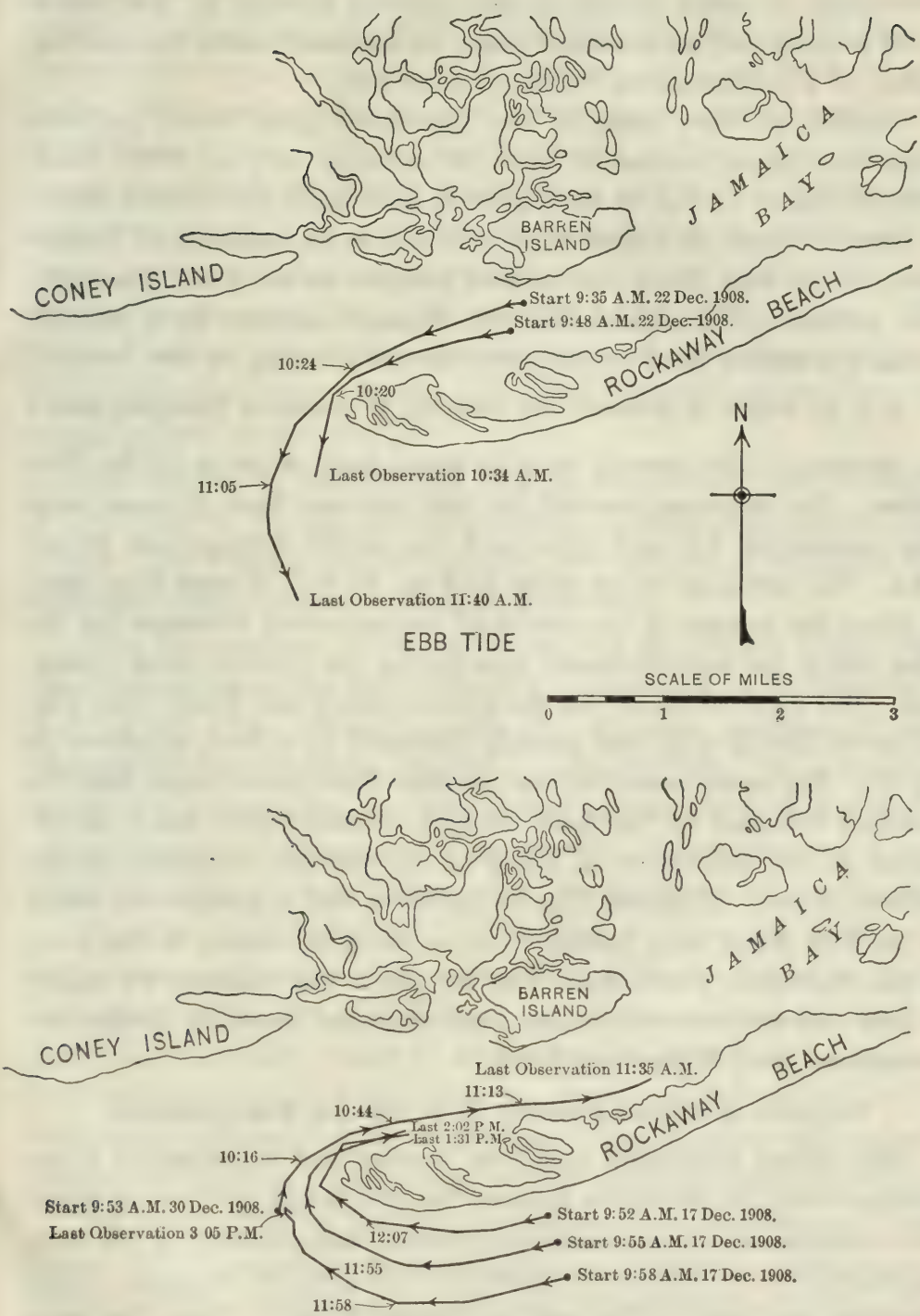
PATH OF A FLOAT IN THE HARLEM RIVER

FIG. 31.



PATH OF A FLOAT IN THE HARLEM RIVER

FIG. 32.



PATHS OF FLOATS THROUGH ROCKAWAY INLET.

AS OBSERVED UNDER THE DIRECTION OF COL. JOHN G.D. KNIGHT, CORPS OF ENGINEERS, U.S.A.

FIG. 33.

observation, in order to change the observed velocity to that which would have existed for a normal range, as discussed under the heading, "Effect of Tidal Range on Velocity," page 1996.

Calculations were made of the theoretical paths which particles would take during successive tides, as explained on page 1988. Such paths are shown in Fig. 34 for a particle starting in the Hudson River, off Grant's Tomb, on a flood tide; and in Fig. 35, starting off College Point, in the East River, one on flood and one on ebb tide. The paths were worked out by calculating the distance traveled by a particle during a complete flood or ebb period by substituting in the formula, $\frac{2}{\pi} A\ t$, in which A denotes the velocity, in knots at strength, and t the duration of the current, in solar hours, both as given in the Tide Tables. The distances traveled on both ebb and flood currents were thus determined for each mile, and the results arranged in proper order. The plottings of the paths in Figs. 34 and 35 were then made by using the average of the results of the calculated distances for the miles which the particle would pass during the current being considered. The path calculated for the Hudson River and Upper Bay, Fig. 34, agrees closely with that actually traversed by a float, as shown in Fig. 22. The paths calculated for the East River were longer than the distances traversed by the floats on each current, which was to be expected, as the calculation is based on the particle remaining in the swiftest thread. However, Fig. 35 indicates that a particle set adrift in the East River near College Point is likely to remain in the river, as was the result with a float, Fig. 28, which was followed for nearly 3½ days and never passed farther south than the Brooklyn Bridge, nor farther east than Whitestone Point.

Volumes of Water Flowing on Flood and Ebb Currents.

The volume flowing on each tidal current can be estimated in accordance with the principles deduced for calculating the flow of rivers. The estimates will be approximately correct, and, considering the daily variations, will be sufficiently accurate for all practical purposes. These estimates are based on the area of a selected section below the mean level of the water surface, the mean velocity of the current, and the time during which the current flows.

It is neither a simple nor an easy matter to make gaugings of the volumes flowing on the tidal currents, because the volumes are so large,

and the measurements would have to extend over long periods, in order to average out the daily variations; and they would have to be made at stations located so closely together as to interfere with navigation. For exact results, current velocity measurements would have to be taken at many different depths at each station. Such measurements should be made nearly coincident at each station, and repeated at short intervals of time, because the velocities of the threads at different depths may vary, and are subject to changes in direction which are not synchronous, due to the reversals of the tidal currents. The work of making actual gauge measurements of such large volumes as flow in the East River, the Hudson River, and through The Narrows would be very costly.

The volumes of the currents passing any selected section during a tidal interval are not constant, because of lunar and seasonal variations. The average or mean volumes, however, can be estimated with sufficient accuracy for all practical purposes.

With the exception of the East River and the Harlem River, where the currents are purely hydraulic, that is, produced by a difference in level of the water surfaces at each end, the main currents are complicated by the discharges of the land-waters from the water-sheds above. In consequence, the ebb currents exceed the flood currents in volume by just the discharges of land-water. This is true under mean conditions, that is, when the ranges of consecutive tides are the same.

Therefore, each tidal flow is composed of a periodic portion due to the true tidal wave, and a non-periodic portion due to the land-water discharge. These two portions are mixed into one volume, but, for purposes of estimate, must be kept distinct. It will be clear, then, that the resultant discharge from the harbor to the sea will be the land-water discharges of the rivers, and no more, except the southerly net flow of the East River, which, as will be shown later, is not great.

The rise and fall of the tides and the current velocities may be assumed to vary according to the ordinates of a sinusoid. Any deviation is due to local conditions or to temporary irregularities. The true tidal actions are periodic, and each tide extends over a period of 6 lunar hours or (nearly) 6.21 solar hours.

The U. S. Coast and Geodetic Survey has made estimates of the volumes of the tides flowing past selected sections, for each month, by

SURFACE CURRENT MOVEMENTS DURING SUCCESSIVE TIDES THROUGH THE HUDSON RIVER, UPPER AND LOWER NEW YORK BAYS.

CALCULATED FROM VELOCITIES GIVEN IN TIDE TABLES FOR 1908.

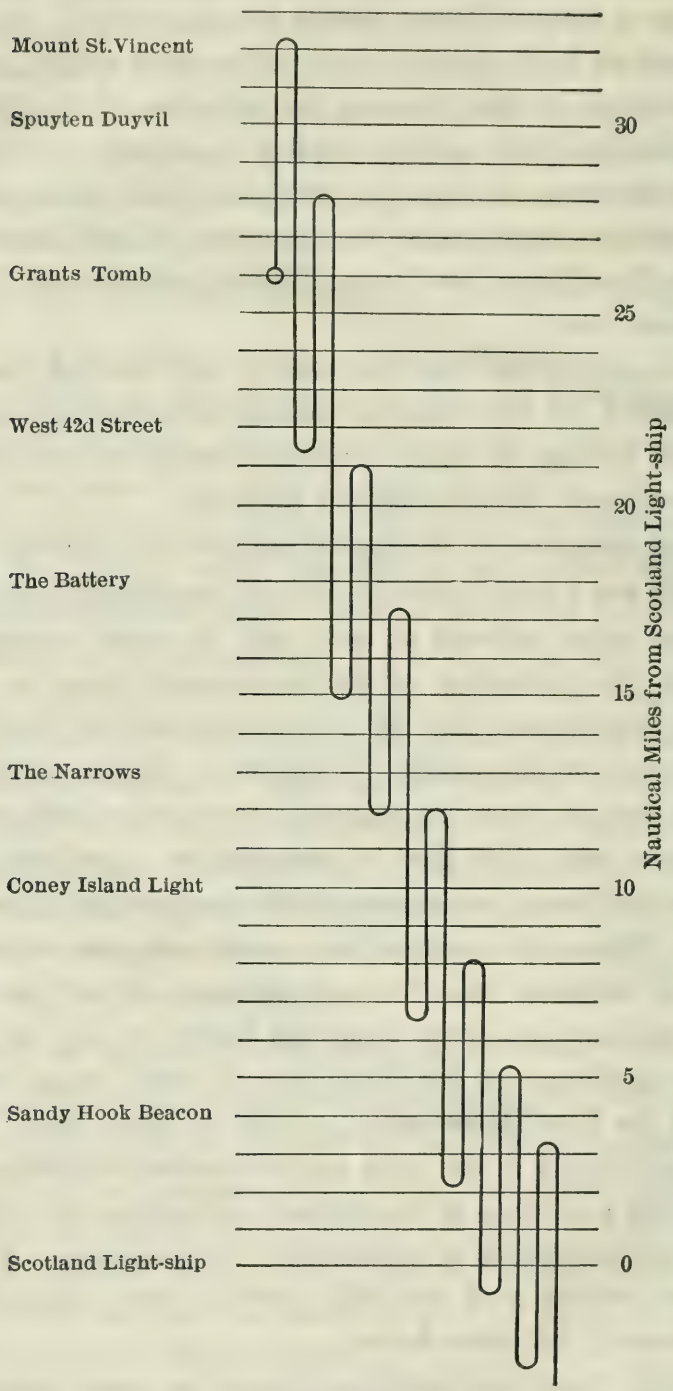


FIG. 34.

SURFACE CURRENT MOVEMENTS DURING SUCCESSIVE TIDES THROUGH THE EAST RIVER, UPPER AND LOWER NEW YORK BAYS.

CALCULATED FROM VELOCITIES GIVEN IN TIDE TABLES FOR 1908.

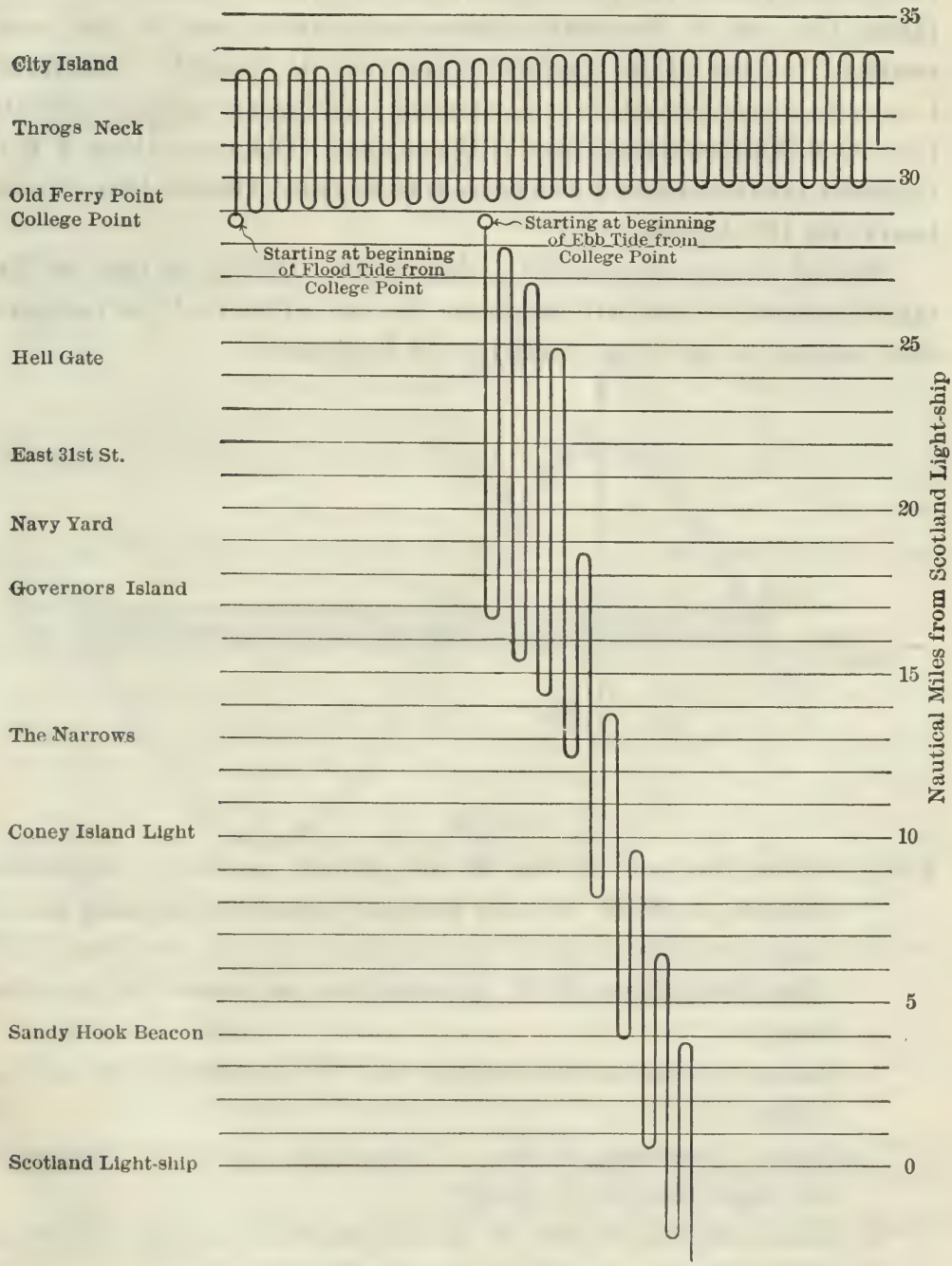


FIG. 35.

using the monthly mean discharge of land-water, and then averaging these monthly results for the yearly mean.

Draw a line, *A B C*, Fig. 36, and divide it into lunar hours. Draw a curve of sines, *A M B C*, such that *M N* represents the maximum rate of flow due to the periodic portion. Its value is found by multiplying the area of the section below mean water level by the mean sectional velocity of the current at the time of strength. The result is usually given in cubic feet per second. The mean sectional velocity is about 0.75 times the velocity at the surface. Distances along *A B C* represent lunar hours and also degrees of angles. Thus *A B* is 6 lunar hours and 180 degrees.

Parallel to, and above, *A B C*, draw a line, *D E F*, so that the distance between the two will represent the rate of flow of the non-periodic portion, or the river discharges of land-water.

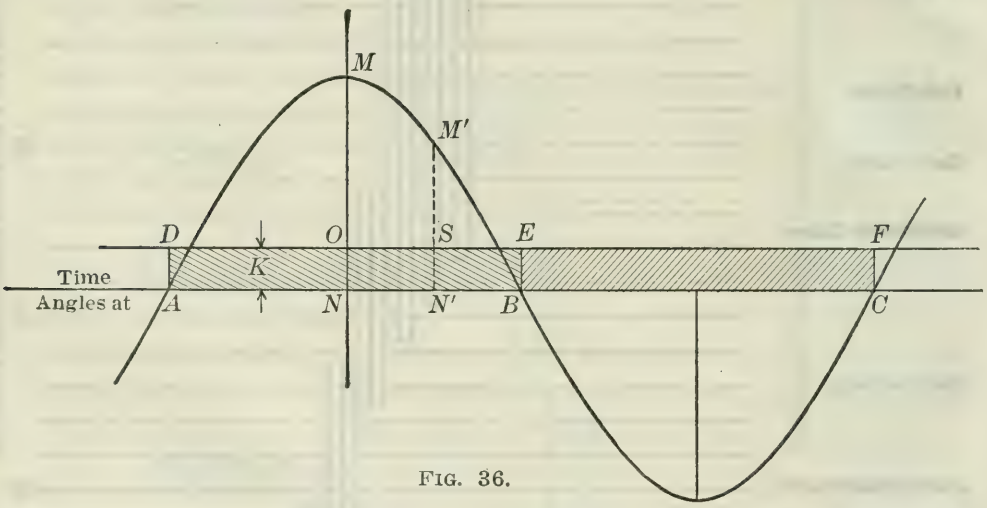


FIG. 36.

Let *R* denote the rate of flow of the periodic portion at maximum strength, or *M N*, in cubic feet per second (or per solar hour);

" *R'* denote the rate of flow of the periodic portion at any time, *N N'*, after strength, or *M' N'*, in cubic feet per second (or per solar hour);

" *t* denote the time after strength, or *N N'*, in seconds (or in solar hours);

" *a* denote a constant, so that *a t* equals the number of degrees in the angle between *N* and *N'*;

" *K* denote the rate of flow of the non-periodic portion, or *O N*, in cubic feet per second (or per solar hour);

" θ denote the number of degrees in the angle between *O* and *E*, or where *R'* equals *K*.

Then:

$M'\ N' = R' = R \cos.\ a\ t$

$M'\ S = y = R \cos.\ a\ t - K.$

As θ is the angle at which $R \cos.\ a\ t = K$, $\cos.\ \theta = \frac{K}{R}$.

$O\ E =$ half duration of flood $= \frac{\theta}{a}$.

Therefore,

the area, $O\ M\ M'\ E = \int_0^\theta y\ dt = \int_0^\theta (R \cos.\ a\ t - K)\ dt,$

or, the volume of half flood $= \int_0^\theta R \cos.\ a\ t\ dt - K\ dt$

$$= \left[{}_0^\theta \frac{R}{a} \sin.\ a\ t - Kt \right]$$

$$= \frac{R}{a} \sin.\ \theta - K\frac{\theta}{a} - \frac{R}{a} \sin.\ O + K\frac{O}{a} \ldots\ldots\ldots (1)$$

As $\sin.\ \theta = \sqrt{1 - \cos.^2 \theta}$, or $\sqrt{1 - \frac{K^2}{R^2}}$, and as θ is the angle the cosine of which is $\frac{K}{R}$, Equation (1) becomes

$$= \frac{R}{a}\sqrt{1 - \frac{K^2}{R^2}} - K\frac{\cos.^{-1}\frac{K}{R}}{a}$$

$$= \frac{1}{a}\left[R\sqrt{1 - \frac{K^2}{R^2}} - K \cos.^{-1}\frac{K}{R} \right] \ldots\ldots\ldots\ldots (2)$$

For time in seconds,

$$a = \frac{\pi}{\text{number of seconds in 6 lunar hours}} = \frac{\pi}{22\,357},$$

or, for solar hours,

$$a = \frac{\pi}{\text{number of solar hours in 6 lunar hours}} = \frac{\pi}{6.21};$$

therefore, the volume flowing on the total flood is twice Equation (2).

After substituting the value of a, for the time in seconds, the total flood volume

$$= 14\,233 \left[R\sqrt{1 - \frac{K^2}{R^2}} - K \cos.^{-1}\frac{K}{R} \right] \ldots\ldots\ldots\ldots (3)$$

The volume flowing on the total ebb is the flood volume plus the run-off for 12 lunar hours. This latter quantity is represented by the area, $A\ B\ C\ F\ E\ D$, or by $\frac{2\ \pi\ K}{a}$, as $A\ C = \frac{2\ \pi}{a}$.

The volume flowing on the ebb, therefore, is

$$= \frac{2\ K\ \pi}{a} + 14\,233\left[R\sqrt{1 - \frac{K^2}{R^2}} - K \cos.^{-)} \frac{K}{R}\right]$$

$$= 44\,714\ K + 14\,233\left[R\sqrt{1 - \frac{K^2}{R^2}} - K \cos.^{-)} \frac{K}{R}\right] \ldots\ldots (4)$$

Taking a section between Sandy Hook and Coney Island, the value of R is 1 685 812 cu. ft. per sec.,* being the product of the mean cross-sectional area and 0.75 of the average surface velocity. The value of K, or the rate of run-off, is variable for each month, but the monthly means are given in Table 5. Substituting these values in Equations (3) and (4), and working out the volumes of flood and ebb for each month, the mean flood and ebb volumes can be estimated by averaging the monthly results.

The mean total ebb volume is 24 658 499 000 cu. ft.
The mean total flood volume is 23 322 888 000 " "

A section was taken across The Narrows, and Equations (3) and (4) were solved for each monthly result; the details are given in Table 6. In this way the flood and ebb volumes for each month can be ascertained. The mean result for the year has been calculated, and the averages for the 12 months are also given.

The velocities of current at strength have been taken from Table 4. The values of K have been found by adding the resultant flow of the East River, which is about 40 000 000 cu. ft. per 6 lunar hours (page 2041), to the total of the land-water entering the Upper Bay above The Narrows, as given in Table 5, and dividing the sum by 22 357, or the number of seconds in 6 lunar hours.

The mean volumes of the normal tidal flows past any section can be estimated closely by a shorter process: Find the mean height of the curve, AMB, Fig. 36, for both flood and ebb currents; then multiply this average rate of flow by the area of the section and by the duration of the current in time, and the result will be the mean of the volumes

* From information received from the U. S. Coast and Geodetic Survey, January 22d, 1910.

TABLE 6.—Section at The Narrows.

Data: Width = 5 280 ft.
Average depth = 61 ft.
Area of section = 322 000 sq. ft.
Velocity at strength, flood = 1.8 knots.
" " " ebb = 2.1 "
" " " mean = 1.95 " = 3.3 ft. per sec.

R = area × 0.75 × velocity at strength (mean of ebb and flood).
R = 800 000 cu. ft. per sec.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Month.	K	$R\sqrt{1-\frac{K^2}{R^2}}$	Cos. $-)\ \frac{K}{R}$	K cos. $-)\ \frac{K}{R}$	Column 3 minus Column 5.	Column 6 × 14 233. Flood volume.	Column 2 × 44 714.	Column 7 plus Column 8. Ebb volume.
Jan.	21 510	799 712	1.5442	33 216	766 496	10 950 000 000	961 798 000	11 911 798 000
Feb.	17 240	799 824	1.5493	26 710	773 114	11 004 000 000	770 860 000	11 774 860 000
Mch.	57 700	797 912	1.4988	86 481	711 431	10 127 000 000	2 579 998 000	12 706 998 000
Apr.	59 720	797 768	1.4960	89 341	708 427	10 084 000 000	2 670 200 000	12 754 200 000
May.	31 740	799 368	1.5313	48 603	750 765	10 686 000 000	1 419 150 000	12 105 150 000
June.	26 110	799 576	1.5382	40 162	759 414	10 810 000 000	1 167 350 000	11 977 350 000
July.	17 390	799 808	1.5491	26 939	772 869	11 001 000 000	777 530 000	11 778 530 000
Aug.	15 720	799 848	1.5511	24 383	775 465	11 038 000 000	702 870 000	11 740 870 000
Sept.	17 630	799 808	1.5488	27 305	772 503	10 996 000 000	788 270 000	11 784 270 000
Oct.	23 830	799 648	1.5412	36 727	752 921	10 718 000 000	1 065 400 000	11 783 400 000
Nov.	22 180	799 696	1.5431	34 226	755 370	10 752 000 000	991 680 000	11 743 680 000
Dec.	28 010	799 504	1.5360	43 023	756 481	10 768 000 000	1 252 350 000	12 020 350 000
Mean.	28 230	799 496	1.5358	43 356	756 140	10 762 000 000	1 262 200 000	12 024 200 000
Average for 12 months						10 744 500 000		12 006 800 000

flowing through the section on both ebb and flood currents. The flood volume will be the mean, thus found, less one-half of the land-water discharged during 12 lunar hours; and the ebb volume will be the mean volume plus one-half of the land-water.

The writer thus estimated the volumes flowing on the tidal currents past various sections of the harbor, chiefly based on investigations made by the Metropolitan Sewerage Commission. These results agree fairly well with the estimates made by the United States Coast and Geodetic Survey.

For comparison, the tidal volumes at The Narrows are given in cubic feet:

	Ebb.	Flood.
By short method.....	12 041 200 000	10 778 800 000
" Equations (3) and (4).	12 024 200 000	10 762 000 000
" U. S. C. and G. Survey.	12 211 187 000	11 028 853 000

The U. S. Coast and Geodetic Survey results were calculated by Equations (3) and (4), but were based on a mean velocity at strength of 2 knots, with no allowance for any resultant discharge from the East River.

Volumes of Tidal Discharges.

The Narrows.

The section between Fort Wadsworth and Fort Hamilton is 5 280 ft. wide, and its average depth below mean water level is 61 ft., so that its area is 322 000 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.34 knots, and on flood 1.17 knots. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission and drawing a smooth curve through the points. The areas below these approximate sinusoids were then divided by their lengths, giving the average velocities. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.255 knots, or 7 631 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75. The mean cross-sectional velocity, therefore, is 5 723 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 1 837 000 000 cu. ft. per solar hour as the mean rate of flow through the section.

The length of time that each current flows was taken at 6 lunar hours, or 6.21 solar hours. The mean volume flowing on ebb and flood currents is 11 410 000 000 cu. ft., obtained by multiplying the mean rate of flow through the section by the length of time.

The mean yearly discharge of land-water run-off past The Narrows for 6 lunar hours is 591 200 000 cu. ft., or 1 182 400 000 cu. ft. per tidal cycle of 12 lunar hours. These figures are from the records of the U. S. Coast and Geodetic Survey and the New York State Engineer and Surveyor. See Table 5.

In addition to this run-off, there is a net discharge through the East River from the Sound into the Upper Bay which passes out through The Narrows. This net discharge is about 80 000 000 cu. ft. per tidal cycle, as explained on page 2041.

The total net discharge, therefore, is 1 262 400 000 cu. ft.; and, from what has been stated on page 2034, the average ebb volume will be approximately the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of this net discharge, or:

Average volume of ebb current, 12 041 200 000 cu. ft.
Average volume of flood current, 10 778 800 000 " "

Hudson River.

The section of the river opposite West 39th Street, Manhattan, is 3 900 ft. wide, and its average depth below mean water level is 37.44 ft., so that its area is 146 000 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.86 knots, and on flood 1.26 knots. These velocities were obtained by plotting curves from information obtained from the U. S. Coast and Geodetic Survey and from the U. S. Coast Pilot, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.56 knots, or 9 485 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75. The mean cross-sectional velocity, therefore, is 7 113 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 1 038 000 000 cu. ft. per solar hour as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 6 447 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

The mean yearly discharge of land-water run-off past Battery Place for 6 lunar hours is 543 600 000 cu. ft., or 1 087 200 cu. ft. per tidal cycle of 12 lunar hours. These figures are from the records of the U. S. Coast and Geodetic Survey and the New York State Engineer and Surveyor. See Table 5.

From what has been stated on page 2034, the average ebb flow will be the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of this net discharge, or:

Average volume of ebb current, 6 990 600 000 cu. ft.
Average volume of flood current, 5 903 400 000 " "

East River.

The run-off from the areas which naturally drain into the East River is small compared with the volumes of the tidal discharges. As the East River is a strait, the flow is subject to the actions of the separate tidal waves entering at each end, Throgs Neck and Governors Island.

The difference in water level creates a hydraulic flow through the strait or river. As the "westerly and southerly" and the "northerly and easterly" hydraulic slopes are equal, there would be no reason to expect a greater flow in one direction than in the other.

A sufficient number of actual measurements of velocity, and extending over sufficiently long periods of time, have not been made to determine positively the mean volumes flowing on the two tidal currents; therefore, the difference between the volumes can only be estimated by theoretical reasoning.

If a base line is drawn and divided into equal spaces, each of which represents a lunar hour, and if a sinusoid be drawn on this base line to represent the rise and fall of the water above and below mean water level off Willets Point, then each ordinate would represent the surface level of the water above or below mean water level at each lunar hour.

If a second sinusoid be drawn in a similar manner for the rise and fall of the tide at Governors Island, then, by placing it so as to be spaced at the proper interval of time relative to the Willets Point

sinusoid, a comparison can be made of the elevations of the water surface at both ends of the strait for every lunar hour. These curves are represented in Fig. 7, and from a study of them it will be noted that, from the lunar hours of IX to III after the transit of the moon, the surface of the water at Willets Point is above that at Governors Island, and a reverse condition is maintained from the lunar hours of III to IX. These two sinusoids form lunes, the ordinates of which represent the difference of the water levels between the ends of the straits at each lunar hour. If the centers of these lunes are determined, it will be noted that, during the time when the current is flowing from Willets Point to Governors Island, the water level is 0.74 ft. above mean water level. For the reverse current, from Governors Island to Willets Point, the average water level is the same distance below mean water level. In other words, the cross-sectional area of the stream during the "westerly and southerly" flow would be greater than during the "northerly and easterly" flow; or, a greater flow would occur from the Sound to the Upper Bay than in the reverse direction, if the velocities were the same in each direction.

Assuming that the average depth of the river below mean water level is 38 ft., then the average depth during the southerly flow would be

$$38 + 0.74 = 38.74 \text{ ft.},$$

and in a similar way the average depth during the northerly flow would be

$$38 - 0.74 = 37.26 \text{ ft.}$$

If the velocities are the same in each direction, and the lengths of time during which the currents flow are equal, then the mean volumes flowing on a southerly current would be

$$\frac{38.74}{37.26} = 1.04$$

times the volume flowing on a northerly flow. In other words, the southerly discharge of the river would be, from this reasoning, about 4% greater than the northerly discharge.

If, at different sections of the river between Willets Point and the Battery, sinusoids are drawn to represent the surface of the water above and below mean water level, and on these diagrams the northerly and southerly currents are marked, and the average water levels are determined for each current, it will be found that the cross-sectional

area of the river is greater on the southerly current for those sections between Willets Point and some place at about 60th Street; but for the sections between the Battery and about 60th Street, the reverse condition obtains. From this it would seem as if the southerly flow did not exceed the northerly flow by the quantity which would be expected from the difference in water level at each end of the strait.

Observations made by the U. S. Coast and Geodetic Survey indicate that, generally speaking, the current having the larger section has the smaller velocity, and *vice versa,* thus leaving the question of the quantity of net discharge uncertain.

Henry Mitchell* gives, as the result of some of his observations, the volumes passing through a section opposite Wall Street, 4 341 100 000 cu. ft. on flood and 4 383 500 000 cu. ft. on ebb, or a difference of only 1% in favor of the southerly flow.

The Coast Survey's observations for sections between Wallabout Bay and the southern extremity of Blackwells Island indicate† that the flood volume is quite as great as that of the ebb. These observations covered work done at 18 stations. The Survey's observations between Blackwells Island Light and Throgs Neck, taken at 21 stations, indicate† that the easterly current is decidedly stronger than the westerly, but that the average cross-sectional area of the currents is greater on the westerly flow. Thus the westerly net discharge cannot be great, for these figures would indicate a small net discharge easterly into the Sound.

The Survey's observations around Blackwells Island, taken alone, indicate‡ a small net southerly discharge, the west-and-south-going volume being about 3% greater than the north-and-east-going volume. As Blackwells Island is about midway between the ends of the strait, and as the stream is well defined at this point, these observations seem to be important in deciding the question of whether there is or is not a net westerly and southerly discharge.

Although Henry Mitchell lays great stress on the difference in cross-sectional areas, which at Hell Gate amounts to nearly 8%, and thus reasons that the westerly discharge is about that much greater than the easterly discharge, the theoretical state of the conditions indicates

*Report of the U. S. Coast and Geodetic Survey, 1871, p. 131.
†U. S. Coast and Geodetic Survey, Letter of February 6th, 1909.
‡U. S. Coast and Geodetic Survey, February 6th, 1909.

that the volume flowing on the currents will not vary directly as the cross-sections of the two streams at Hell Gate. This is true, because, the smaller the section at the Gate, the greater must be the velocity at that point, as the length through the Gate is a small fraction of the lengths of the other reaches of the river. This result can be shown by Bernoulli's theorem applied to a strait, which becomes, when friction is taken into account,*

$$v^2 = \frac{2\,g\,(\zeta_{,} - \zeta_{,,})}{1 + \zeta' \frac{P_1}{\Omega_1} L_1 + \zeta' \frac{P_2}{\Omega_2} L_2 \left(\frac{\Omega_1}{\Omega_2}\right)^2 + \zeta' \frac{P_3}{\Omega_3} L_3 \left(\frac{\Omega_1}{\Omega_3}\right)^2 + \ldots}$$

in which $\zeta_{,}$ and $\zeta_{,,}$ denote the heights of the surface of the water, at the Gate and beyond, above half-tide level; ζ' a friction factor; P the wetted perimeter; Ω the area of the section, and L the length of the channel. The subscript, 1, refers to the Gate, and subscripts, 2, 3, etc., refer to other portions of the channel. The resistance, as expressed in the denominator, increases if the cross-section at the Gate, Ω_1, increases, the other sections, Ω_2, Ω_3, etc., remaining the same. The velocity then would vary inversely as Ω_1. Because the other sections, Ω_2, Ω_3, do not remain the same for both streams, the real velocity will vary less than changes in Ω_1.

Taking into account the information thus briefly outlined, for the whole length of the river, it "seems probable that the volume transmitted westerly can be not more than one or two per cent. greater than the volume transmitted easterly."†

As the volume flowing on each tidal current is about 4 000 000 000 cu. ft., the net or resultant flow from the Sound to the Upper Bay would be, at 2%, 80 000 000 cu. ft. per tidal cycle.

East River at Brooklyn Bridge.—The section of the river at Brooklyn Bridge is 1 500 ft. wide, and its average depth below mean water level is 44.20 ft., so that its area is 66 300 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 2.42 knots, and on flood 2.22 knots. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 2.32 knots, or 14 105 ft. per solar hour.

* See Appendix C.

† Letter, U. S. Coast and Geodetic Survey, August 14th, 1908.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows and Hudson River. The mean cross-sectional velocity, therefore, is 10 504 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 696 500 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 4 330 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

On page 2041 it is shown that there is a net discharge through the East River from the Sound into the Upper Bay, and that it is about 80 000 000 cu. ft. per tidal cycle. This figure includes the mean yearly discharge of land-water run-off, which amounts to about 3 400 000 cu. ft. per tidal cycle of 12 lunar hours. From what has been stated on page 2034, the average ebb flow will be the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of this net discharge, or:

Average volume of ebb current, 4 370 000 000 cu. ft.
Average volume of flood current, 4 290 000 000 " "

East River off 11th Street.—The section of the river off East 11th Street, Manhattan, is 2 235 ft. wide and its average depth below mean water level is 36.13 ft., so that its area is 80 750 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.91 knots, and on flood 1.85 knots. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.88 knots, or 11 430 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows and Hudson River. The mean cross-sectional velocity, therefore, is 8 572 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 692 200 000 cu. ft. per solar hour, as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 4 299 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

As shown on page 2041, the net discharge toward the Upper Bay is about 80 000 000 cu. ft. per tidal cycle. The average volumes of ebb and flood currents were obtained in the same manner as those for the East River at Brooklyn Bridge.

Average volume of ebb current, 4 339 000 000 cu. ft.
Average volume of flood current, 4 259 000 000 " "

East River off 31st Street.—The section of the river off East 31st Street, Manhattan, is 3 180 ft. wide, and its average depth below mean water level is 39.07 ft., so that its area is 124 250 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.13 knots, and on flood 1.29 knots. These velocities were obtained by averaging a number of observations, made near this section by the U. S. Coast and Geodetic Survey, of the surface velocities at strength. It was assumed that the velocities vary as ordinates of a sinusoid, and the average velocities were obtained by multiplying the velocities at strength by $\frac{2}{\pi}$.

The mean of the ebb and flood average velocities of the swiftest surface thread is 1.21 knots, or 7 357 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows, Hudson River, and the other sections on the East River. The mean cross-sectional velocity, therefore, is 5 518 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 685 600 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 4 258 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

As shown on page 2041, the net discharge toward the Upper Bay is about 80 000 000 cu. ft. per tidal cycle. The average volumes of ebb and flood currents were obtained in the same manner as those for the East River at Brooklyn Bridge.

Average volume of ebb current, 4 298 000 000 cu. ft.
Average volume of flood current, 4 218 000 000 " "

*East River off 81st Street.**—The west channel is 800 ft. wide, and its average depth below mean water level is 39 ft., so that its area

* The data for this estimate are from a letter of the U. S. Coast and Geodetic Survey dated August 14th, 1908.

is 31 200 sq. ft. Similar figures for the east channel, a little southward from Graham Avenue are 600, 26.5 and 15 900, respectively. In the former section the maximum surface velocity is 4.9 knots and in the latter 4.0 knots, according to observations made in 1857 and 1874. Multiplying these velocities by 0.75 to reduce them to cross-sectional velocities at time of strength, and by $\frac{2}{\pi}$ to reduce them to mean cross-sectional velocities during each tide, and by the area of the section, the following volumes were obtained:

2 756 295 600 cu. ft. passing western section.
1 146 685 915 " " " eastern "

The sum of these two volumes is 3 902 981 515 cu. ft., which is the mean volume flowing on ebb and flood currents. Dividing this figure into the average volume flowing on ebb and on flood, in the same manner as for the East River off Brooklyn Bridge, the following figures were obtained:

Average volume of ebb current, 3 943 000 000 cu. ft.
Average volume of flood current, 3 863 000 000 " "

East River off Old Ferry Point.—The section of the river off Old Ferry Point is 3 000 ft. wide, and its average depth below mean water level is 43.88 ft., so that its area is 131 650 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 0.88 knot, and on flood 1.09 knots. These velocities were obtained by averaging a number of observations of the surface velocities at strength at this point, made by the U. S. Coast and Geodetic Survey. Assuming that the velocities vary as the ordinates of a sinusoid, the average surface velocity at strength was multiplied by $\frac{2}{\pi}$ to obtain the average velocity of the swiftest surface thread. The mean of the ebb and flood average velocities of the swiftest surface thread is 0.985 knot or 5 989 ft. per solar hour.

The coefficient to reduce the surface velocity to cross-sectional velocity was taken at 0.75, the same as for The Narrows, Hudson River, and the other sections on the East River. The mean cross-sectional velocity, therefore, is 4 491 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 591 300 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 3 672 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

As shown on page 2041, the net discharge toward the Upper Bay is about 80 000 000 cu. ft. per tidal cycle. The average volumes of ebb and flood currents were obtained in the same manner as those for the East River at Brooklyn Bridge.

Average volume of ebb current (westerly), 3 712 000 000 cu. ft.
Average volume of flood current (easterly), 3 632 000 000 " "

Reason for the Volumes of Flow Diminishing from the Southerly End to the Easterly End of the East River.

The average volumes of flow for the ebb and flood currents at different sections of the East River, in cubic feet, are as follows:

	Ebb.	Flood.
Off Brooklyn Bridge.....	4 370 000 000	4 290 000 000
Off East 11th Street.....	4 339 000 000	4 259 000 000
Off East 31st Street.....	4 298 000 000	4 218 000 000
Off East 81st Street.....	3 943 000 000	3 863 000 000
Off Old Ferry Point.....	3 712 000 000	3 632 000 000

It will be seen from this statement that the volumes of flow decrease from the southerly end to the easterly end of the river. A reason for this decrease was sought, with the following result:

On Fig. 37 curves are drawn for each lunar hour, with distances as abscissas and velocities as ordinates, flood velocities being plotted above, and ebb velocities below, the datum lines. Another curve is plotted to the right showing the average elevation of the water in the East River for each lunar hour. This latter curve is cross-hatched for the times when the water is rising, and is left blank when falling.

It will be noted that, during the 6 lunar hours when the average level of the water is rising, practically for the entire length of the river, the current is flowing flood (north and east). In order that the level of the water may rise, it is necessary for more water to flow in at the Battery than flows out at Throgs Neck, in order to make up this rise. In the same way, for the 6 lunar hours when the average elevation of the water is falling, the current is ebb (flowing west and south) for practically the entire length of the stream.

In order that the elevation of the water may fall, it is necessary that a greater quantity should flow out of the river at the Battery than flows in at Throgs Neck.

In discussing this subject, it must be kept clearly in mind that the velocities of the currents are due to the heads produced by the varying hydraulic slopes, and that the rise and fall of the water are due to the progress of the tidal waves entering the East River from each end.

The advance of the tidal waves is retarded by the shallowness of the waters and by the natural obstructions. The tidal wave entering the East River at Governors Island is about 3 hours, 5 min. ahead of the wave entering at Throgs Neck. As these waves progress through the river in opposite directions, there is a superposition and a consequent interference. This interference and the natural obstructions of the river have the effect of making the wave vary from a true sinusoid.

As the Governors Island wave enters the river before the Throgs Neck wave, greater quantities of water have to flow past points on the lower reaches of the river than points farther up (northerly and easterly) so as to provide for the tidal prism, for, if not, there would be some point in the river (near its mid-length) where the water would not rise or fall.

Kill van Kull.

Kill van Kull off Port Richmond.—The section of the river off Port Richmond is 1 380 ft. wide, and its average depth below mean water level is 28.30 ft., so that its area is 39 060 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.48 knots, and on flood 1.24 knots. These velocities were obtained by plotting the results from the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.36 knots, or 8 269 ft. per solar hour.

The coefficient to reduce the surface velocity to cross-sectional velocity was taken at 0.75, the same as for The Narrows, Hudson River, and East River. The mean cross-sectional velocity, therefore, is, 6 202 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 242 200 000 cu. ft., as the mean rate of flow through the section.

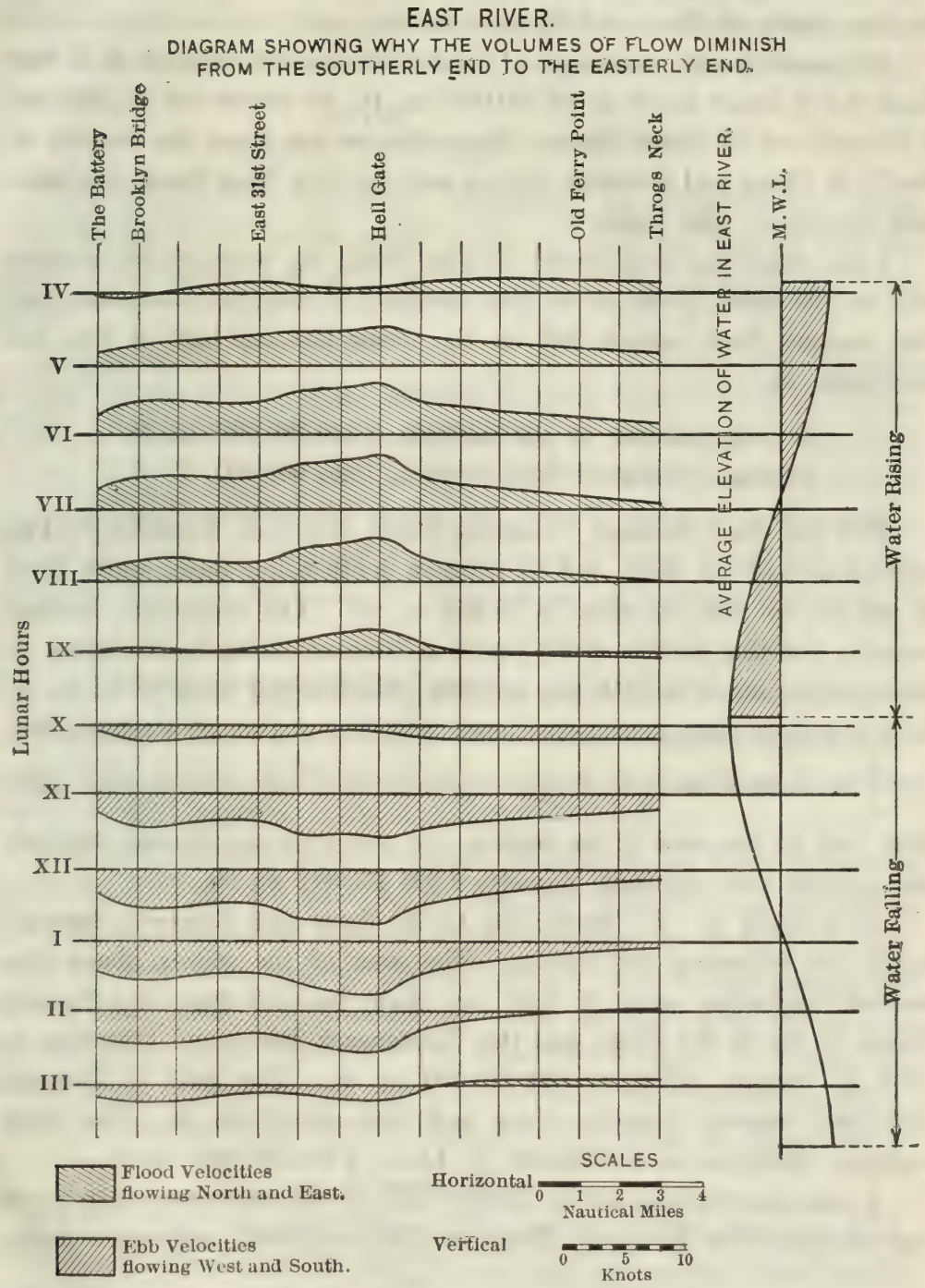

EAST RIVER.
DIAGRAM SHOWING WHY THE VOLUMES OF FLOW DIMINISH FROM THE SOUTHERLY END TO THE EASTERLY END.
The Battery
Brooklyn Bridge
East 31st Street
Hell Gate
Old Ferry Point
Throgs Neck
AVERAGE ELEVATION OF WATER IN EAST RIVER.
M.W.L.
Water Rising
Water Falling
Lunar Hours
IV
V
VI
VII
VIII
IX
X
XI
XII
I
II
III
Flood Velocities flowing North and East.
Ebb Velocities flowing West and South.
SCALES
Horizontal
0 1 2 3 4
Nautical Miles
Vertical
0 5 10
Knots

FIG. 37.

The mean volume flowing on ebb and flood currents is 1 502 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

The mean yearly discharge of land-water run-off through Kill van Kull for 6 lunar hours is 44 100 000 cu. ft., or 88 200 000 cu. ft. per tidal cycle of 12 lunar hours. These figures are from the records of the U. S. Coast and Geodetic Survey and the New York State Engineer and Surveyor. See Table 4.

From what has been stated on page 2034, the average ebb volume will be the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of this net discharge, or:

Average volume of ebb current, 1 546 100 000 cu. ft.
Average volume of flood current, 1 457 900 000 " "

*Kill van Kull Between Constable Point and New Brighton.**—The section is 1 425 ft. wide, and its average depth below mean water level is 26 ft., so that its area is 38 475 sq. ft. The maximum surface velocity for this section is 2.3 knots at time of strength, according to observations made in 1856 and in 1885. Multiplying these velocities by 0.75 to reduce them to cross-sectional velocities at time of strength, and by $\frac{2}{\pi}$ to reduce them to mean cross-sectional velocities during each tide, and by the area of the section, the mean volume flowing through the section was obtained, namely, 1 595 475 341 cu. ft.

As a check on this figure, the U. S. Coast and Geodetic Survey* made the following calculation: The area of the waters above this section, including most of Kill van Kull, Newark Bay, the Passaic River as far as the Falls, and the Hackensack River and branches, is 15.3 sq. statute miles, or 426 539 520 sq. ft. The level of Newark Bay and near-by branches rises and falls about 4.5 ft. The tidal volume, therefore, is $426\,539\,520 \times 4.5 = 1\,919\,427\,840$ cu. ft.

A few observations in the Arthur Kill off Elizabethport, just above the mouth of the Elizabeth River, in 1856 and 1885, indicate a maximum surface velocity of 1.8 knots, or $\frac{18}{23}$ of the value observed in the

* These calculations are from a letter of the U. S. Coast and Geodetic Survey, dated August 14th, 1908.

Kill van Kull. The section of the Arthur Kill is about 600 × 16 = 9 600 sq. ft., which would be equivalent to $\frac{18}{23}$ of 9 600, or 7 513 sq. ft., if the velocity were 2.3 knots, instead of 1.8 knots. Hence, the proportion of the volume flowing through the mouth of the Kill van Kull would be

$$\frac{38\ 475}{38\ 475 + 7\ 513} = 0.8366313.*$$

This proportion, 0.8366313, multiplied by 1 919 427 840, gives 1 605 853 409 cu. ft. for the mean volume entering and leaving Kill van Kull. This agrees well with the results from the determination first given.

Averaging the two figures given by the U. S. Coast and Geodetic Survey for the flow of water through Kill van Kull between Constable Point and New Brighton, 1 600 000 000 cu. ft. is obtained as the mean volume flowing on ebb and flood currents. This figure was divided into volumes flowing on ebb current and flood current in the same manner as for Kill van Kull off Port Richmond, using 88 200 000 cu. ft. per tidal cycle for the mean yearly discharge of land-water run-off.

Average volume of ebb current, 1 644 100 000 cu. ft.
Average volume of flood current, 1 555 900 000 " "

The tidal prism (or the area multiplied by the mean range of tide) between the section off Port Richmond and the section from Constable Point to New Brighton is about 100 000 000 cu. ft.

The mean volume flowing through the section off Port Richmond would be less than the quantity flowing past the section at Constable Point by the tidal prism between these two places, as that much water would be used in increasing the level of the water. Subtracting the tidal prism from the flow through the section, 1 600 000 000 — 100 000 000 = 1 500 000 000 cu. ft. This figure agrees well with the mean volume flowing on ebb and flood currents, 1 502 000 000, as calculated for Kill van Kull, off Port Richmond, as stated on page 2048.

Harlem River.

Harlem River off 144th Street.—The section of the river off West 144th Street, Manhattan, is 460 ft. wide and its average depth below mean water level is 15.67 ft., so that its area is 7 210 sq. ft.

* The proportion flowing through the Arthur Kill would be 0.1633687.

The average velocity of the swiftest surface thread during the tidal period on ebb is 0.71 knot, and on flood 0.69 knot. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 0.70 knot, or 4 256 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows, Hudson River, East River, and Kill van Kull. The mean cross-sectional velocity, therefore, is 3 192 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 23 010 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 142 900 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

At 144th Street the average water level during ebb (northerly) current is 0.98 ft. above mean water level, and during flood (southerly) current is 0.98 ft. below mean water level. The average area of cross-section during ebb current, therefore, is the mean area of the section plus the width multiplied by 0.98, or 7 660 sq. ft.; and the average area of cross-section during flood current is the mean area of the section minus the width multiplied by 0.98, or 6 760 sq. ft. The mean volume flowing on ebb and flood was divided in proportion to the average area of cross-sections on the ebb current and on the flood current.

Average volume of ebb current, 151 800 000 cu. ft.
Average volume of flood current, 134 000 000 " "

Harlem River 600 Feet North of High Bridge.—The section of the river 600 ft. north of High Bridge is 410 ft. wide, and its average depth below mean water level is 11.71 ft., so that its area is 4 800 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.30 knots, and on flood 1.22 knots. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.26 knots, or 7 661 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows, Hudson

River, East River, and Kill van Kull. The mean cross-sectional velocity, therefore, is 5 746 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 27 580 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 171 200 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

At this section the average water level during ebb (northerly) current is 0.58 ft. above mean water level, and during flood (southerly) current is 0.58 ft. below mean water level. The average area of cross-section during ebb current, therefore, is the mean area of the section plus the width multiplied by 0.58, or 5 040 sq. ft.; and the average area of cross-section during flood current is the mean area of the section minus the width multiplied by 0.58, or 4 560 sq. ft. The mean volume flowing on flood and ebb was divided in proportion to the average area of cross-sections on the ebb current and on the flood current.

Average volume of ebb current, 179 800 000 cu. ft.
Average volume of flood current, 162 600 000 " "

Reason for the Volumes of Flow Diminishing from the Northerly End to the Southerly End of the Harlem River.

Although it was not possible to calculate accurately the flow of water in the Harlem River at places other than 144th Street and 600 ft. north of High Bridge, because enough float observations were not made at other points, it is apparent that the quantity of water flowing past any section decreases from the northerly to the southerly end.

The float observations by the Metropolitan Sewerage Commission near Wards Island, in the Harlem River, show small velocities and a tendency of the floats to eddy and drift toward the shore.

The few float observations in the Harlem River near Spuyten Duyvil show velocities considerably greater than those at the southerly end.

Fig. 38 was made to show why the volumes of flow diminish from the northerly to the southerly end. This diagram is similar in construction to Fig. 37 for the East River. The distance from Spuyten Duyvil to Wards Island is laid off for the lunar hours as abscissas and the velocities as ordinates. The flood velocities, or those flowing south,

are laid off to scale below the datum lines, and the ebb velocities, or those flowing north, are laid off to scale above the datum lines.

A curve is drawn at the left to show the average elevation of the water in the Harlem River at each lunar hour. This curve is cross-hatched for the times when the water is rising, and is left blank when falling.

It will be noted that, during the 6 lunar hours when the average level of the water is rising, the current is flowing flood for the entire length of the river. In order that the level of the water may rise, it is necessary that a greater quantity should flow in through Spuyten Duyvil than flows out at Wards Island, to make up this increase of elevation.

In the same way, for the 6 lunar hours during which the average level of the water is falling, the current is flowing ebb for the entire length of the river. In order that the level of the water may fall, it is necessary that more water flow out through Spuyten Duyvil than flows in at Wards Island. If the same quantities should pass Wards Island and Spuyten Duyvil during these 6 lunar hours, the average elevation of the water in the river would not fall, but would remain practically constant. The converse is also true. This is the reason that the volumes of flow diminish from the northerly to the southerly end of the Harlem River.

Resultant Flows.

The volumes flowing on ebb currents exceed those flowing on flood currents, so that there is a resultant flow during each tidal period through the harbor toward the sea.

In the rivers discharging into the harbor, the mean resultant flow equals the mean fresh-water run-off from the water-sheds above. When the ranges (rise and fall) of two successive tides are equal, a quantity equal to the fresh-water run-off must flow out and not return, so as to keep the elevation of the water the same on the next tide. As the ranges change from tide to tide, this will not be true in each individual instance, but will be true under mean conditions.

The following resultant flows are true only for mean conditions; this is, when the range of tide is normal and the mean quantity of land-water run-off is flowing. They are the difference between the flood and ebb discharges, as shown in the preceding paragraphs.

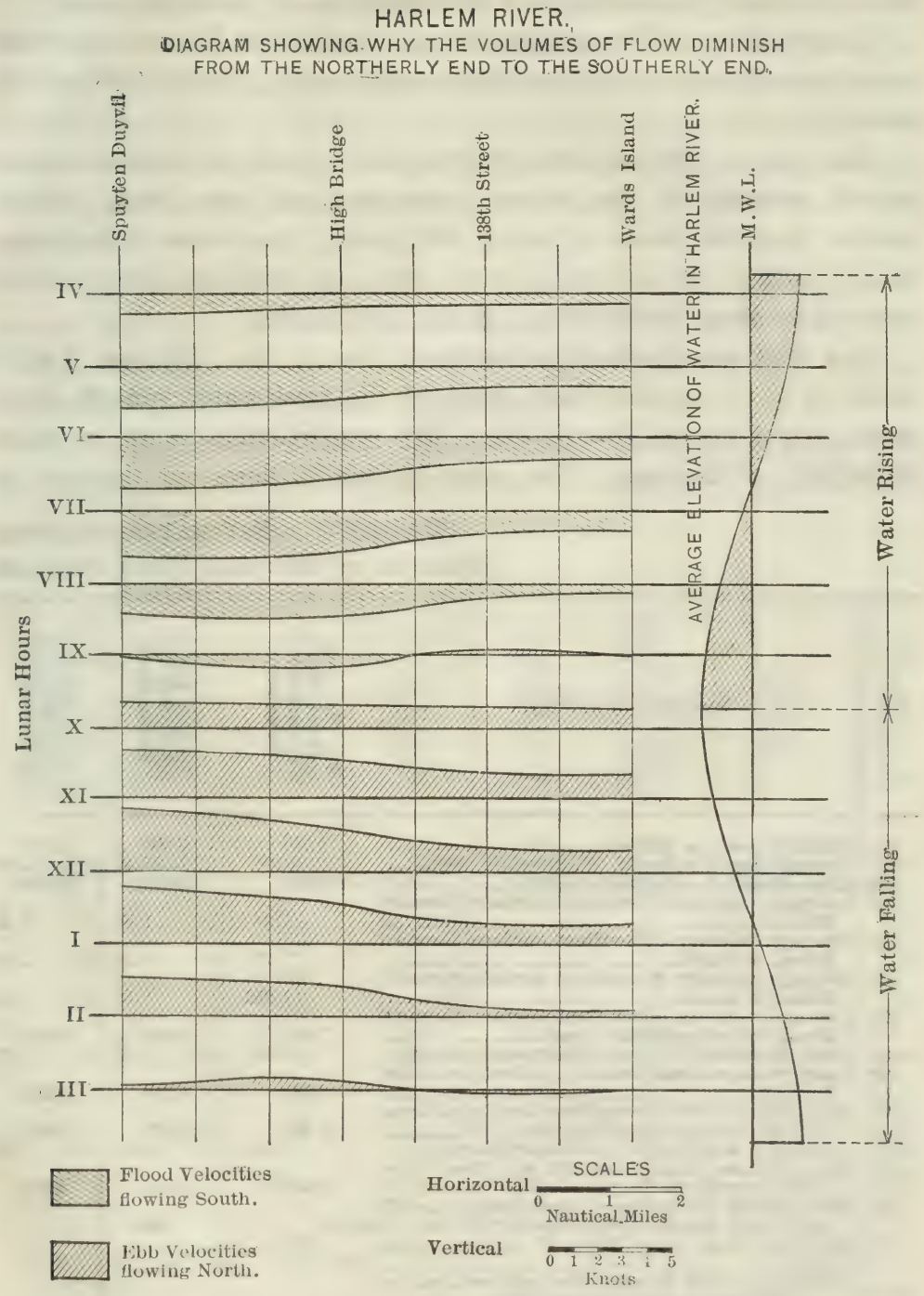

HARLEM RIVER.
DIAGRAM SHOWING WHY THE VOLUMES OF FLOW DIMINISH FROM THE NORTHERLY END TO THE SOUTHERLY END.
Spuyten Duyvil
High Bridge
138th Street
Wards Island
M.W.L.
AVERAGE ELEVATION OF WATER IN HARLEM RIVER.
Water Rising
Water Falling
Lunar Hours
IV
V
VI
VII
VIII
IX
X
XI
XII
I
II
III
Flood Velocities flowing South.
Ebb Velocities flowing North.
SCALES
Horizontal
0 1 2
Nautical Miles
Vertical
0 1 2 3 4 5
Knots

FIG. 38.

The Narrows.—The resultant flow through The Narrows is made up of the resultant flows of the Hudson River, the East River, and the Kill van Kull. It amounts to 1 262 400 000 cu. ft. per tidal cycle.

Hudson River.—The resultant flow in the Hudson River is the fresh-water run-off from the water-sheds above, namely, 1 087 200 000 cu. ft. per tidal cycle.

East River.—The resultant flow in the East River is small, and is caused probably by the average cross-sectional area being slightly greater when the water is flowing ebb (south) than when it is flowing flood (north). As shown on page 2041, the resultant flow (south) amounts to about 80 000 000 cu. ft. per tidal cycle.

The Kill van Kull.—The resultant flow in the Kill van Kull is made up of its proportionate share of the fresh-water run-off of the rivers tributary to Newark Bay. By proportionate share is meant 83.66313% of the total. The other 16.33687% flows out through the

TABLE 7.—ESTIMATES OF TIDAL Volumes of Ebb and Flood Currents,

No.	Elements of Problem.	The Narrows between Forts Wadsworth and Hamilton.	Hudson River off W. 39th St., Manhattan.	East River off Brooklyn Bridge.
(1)	Width of section, in feet	5 280	3 900	1 500
(2)	Average depth below M. W. Level, in feet	61.0	37.44	44.20
(3)	Area of section, in square feet	322 000	146 000	66 300
(4)	Average, velocity of swiftest surface thread: Ebb, in knots	1.34	1.86	2.42
(5)	Average, velocity of swiftest surface thread: Flood, in knots	1.17	1.26	2.22
(6)	Average, velocity of swiftest surface thread: Mean, in knots	1.255	1.56	2.32
(7)	Average, velocity of swiftest surface thread: Mean, in feet per solar hour	7 631.0	9 485	14 105
(8)	Coef., to reduce (7) to cross-sectional velocity.	0.75	0.75	0.75
(9)	Mean cross-sectional velocity, in feet per solar hour	5 723	7 113	10 504
(10)	Mean rate of flow through section, in millions of cubic feet per solar hour (3) × (9)	1 837	1 038	696.5
(11)	Number of solar hours in 6 lunar hours	6.21	6.21	6.21
(12)	Mean volume flowing on ebb and flood currents, in millions of cubic feet (10) × (11)	11 410	6 447	4 330
(13)	Mean yearly discharge of land-water run-off per 6 lunar hours, in millions of cubic feet	591.2	543.6	
	Resultant from East River, 80 million cubic feet	40.0 } 631.2		80
	Average volume flowing through section, in millions of cubic feet per tide:			
(14)	On ebb	12 041.2	6 990.6	4 370
(15)	On flood	10 778.8	5 903.4	4 290
(16)	Resultant flow, excess of ebb over flood, in millions of cubic feet (14) — (15)	1 262.4	1 087.2	80

Arthur Kill (page 2048). The resultant flow through the Kill van Kull amounts to 88 200 000 cu. ft. per tidal cycle.

Harlem River.—The resultant flow in the Harlem River is probably due to the greater average cross-sectional area which obtains on the ebb (northerly flow) than when the flow is in the reverse direction. Calculations gave 17 800 000 cu. ft. off 144th Street, and 17 200 000 cu. ft. at a section 600 ft. north of High Bridge. The mean resultant flow throughout the length of the river is about 17 400 000 cu. ft. per tidal cycle.

Under conditions of maximum net outflow toward the sea, the resultant flow is a little more than twice that under normal conditions. Under conditions of minimum net outflow, the resultant flow is about one-half of that under normal conditions.

In Table 7 are given the various steps by which the ebb and flood discharges, and the resultant flows, were calculated. This table is a

DISCHARGES, IN CUBIC FEET.
Harbor of New York.

East River off E. 11th St., Manhattan.	East River off E. 31st St., Manhattan.	East River off E. 81st St., Manhattan.	East River off Old Ferry Point.	The Kill van Kull off Port Richmond.	The Kill van Kull between Constable Pt. and New Brighton.	Harlem River off 144th St., Manhattan.	Harlem River Section 600 ft. North of High Bridge.
2 235	3 180		3 000	1 380	1 425	460	410
36.13	39.07		43.88	28.30	26	15.67	11.71
80 750	124 250		131 650	39 060	38 475	7 210	4 800
1.91	1.13		0.88	1.48		0.71	1.30
1.85	1.29		1.09	1.24		0.69	1.22
1.88	1.21		0.985	1.36		0.70	1.26
11 430	7 357		5 989	8 269		4 256	7 661
0.75	0.75		0.75	0.75		0.75	0.75
8 572	5 518		4 491	6 202		3 192	5 746
692.2	685.6		591.3	242.2		23.01	27.58
6.21	6.21		6.21	6.21		6.21	6.21
4 299	4 258	3 903	3 672	1 502	1 600	142.9	171.2
				44.1	44.1	Dividing (12) *pro rata* to average areas during currents gives (14) and (15).	
80	80	80	80				
339	4 298	3 943	3 712	1 546.1	1 644.1	151.8	179.8
4 259	4 218	3 863	3 632	1 457.9	1 555.9	134.0	162.6
80	80	80	80	88.2	88.2	17.8	17.2

U. S. C. and G. Survey Letter of August 14th, 1908.

U. S. C. and G. Survey Letter of August 14th, 1908.

summary of the matter contained under the heading, "Volumes of Tidal Discharges," pages 2036 to 2049.

It is self-evident that, under the conditions which obtain for the East and Harlem Rivers, especially the former, there may be great variations from day to day in the resultant flow as given in the preceding paragraphs. It is possible that in the East River the resultant flow on some days may be many times greater than that stated for the normal, and on other days it is possible that it is in the opposite direction, or northerly and easterly. These variations are brought about by various causes, the principal one being the wind inducing abnormal tides in the Upper Bay and in Long Island Sound.

Volumes of Tidal Flow Into and Out of the Upper Bay.

Taking the Upper Bay as a central basin, the entrances and exits are the East River, Hudson River, Kill van Kull, and The Narrows. Into the Upper Bay there is a discharge of water from the Hudson and a proportion of the discharge of the Hackensack and Passaic Rivers, as well as the resultant flow from the Sound through the East River. This resultant flow through the East River toward the Upper Bay may or may not be great, but, under normal conditions, there is a small resultant discharge into the Bay. In consequence, there is a net flow of water seaward through The Narrows, and the effect of this seaward flow is felt possibly some 60 miles off Sandy Hook.*

While the water is flowing into and out of the Upper Bay during a full tidal cycle, the volume of water in the Upper Bay is changing, as is shown by the rise and fall of its water surface. In order to create this rise, more water must flow into the Upper Bay during approximately 6 lunar hours than runs out during the same hours. The converse also is true. As the water surface of the Upper Bay is nearly 21 sq. miles, and the mean range of tide is 4.4 ft., the tidal prism and also the volume of tidal change between any two periods of time can be estimated.

The rates or volumes of flow, in millions of cubic feet per lunar hour, are shown graphically by Fig. 8. To determine the volumes of flow through any one of the entrances to the Upper Bay at any given lunar hour, measure the ordinate to the proper curve in accordance

* U. S. Coast and Geodetic Survey, Vol. VII, 1859-60, Appendix 26.

with the scale marked at the side of the figure. These rates of flow are those under mean conditions, and are sufficiently accurate for all practical purposes.

Fig. 8 was constructed by estimating that the rates of flow vary as the ordinates of a curve of sines. Therefore, for each tidal flow, a sine curve was drawn to scale, so that its length would represent the number of lunar hours that the current would flow on either ebb or flood; and its area, between the curve and the base line, would represent the volumes of flow transmitted, as estimated under the heading, "Volumes of Tidal Discharges," page 2036. Dividing any volume of tidal discharge by six will give the average flow for one lunar hour. Multiplying this result by $\pi \div 2$, which is the ratio that the maximum ordinate of a sine curve bears to its average ordinate, will give the maximum flow for the current selected. This point was plotted to scale, and a sine curve was drawn through it. This work was repeated for each branch of the harbor.

In drawing the curves on Fig. 8 the lunar times were advanced by the number of minutes that the tidal waves required to progress from the Upper Bay to the sections selected for estimating the volumes of flow.

The rates of flow, in millions of cubic feet per lunar hour, were calculated, and the results are given in Table 9 for each branch of the harbor. The results for each lunar hour were obtained by multiplying the maximum flow for the current selected, as determined previously, by the cosine of the angle, which represents in degrees the difference between the lunar time of maximum strength and the lunar time for which the calculation was made.

By way of illustration, the calculation for the first lunar hour is given in Table 8.

The figures in Table 9 are the calculated rates of flow at each lunar hour. The rates of flow are not the volumes flowing throughout a full lunar hour, although they approximate to the latter. In consequence, the columns add up only approximately to the ebb and flood discharges, as given in the preceding paragraphs.

The quantity of water flowing in at any lunar hour should equal that flowing out, when allowance is made for the tidal prism in the Upper Bay. The accuracy of the work is shown by taking the algebraic sum of the ordinates in Fig. 8, which should be zero,

or by adding the horizontal lines at each lunar hour in Table 9, which should give equal quantities of water flowing into and out of the Upper Bay. The error is small, considering the assumptions made, in view of the lack of actual current observations extending over long periods of time.

TABLE 8.—For Lunar Hour I.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Lunar time of maximum strength, water flowing into Upper Bay.	Lunar hours before I, lunar time.	Equivalent, in degrees.	Cosine of Column 4.	Volume flowing, in millions of cubic feet on ebb tide.	Maximum rate of flow, in millions of cubic feet per lunar hour on ebb. Column 6 $\times \frac{\pi}{6} \times \frac{1}{2}$	Rate of flow, in millions of cubic feet per lunar hour at 1 o'clock, lunar time. = Column 5 × Column 7
The Narrows	VI. 7	6.3	189	— 0.9877	12 041	3 153	— 3 114
Hudson River	I. 8	11.2	336	+ 0.9135	6 991	1 830	+ 1 672
East River	XII. 7	0.3	9	+ 0.9877	4 370	1 143	+ 1 129
Kill van Kull	X. 8	2.2	66	+ 0.4067	1 644	430	+ 175
Increase or decrease of water in Upper Bay	X. 8	2.2	66	+ 0.4067	2 530	663	+ 270
							+ 3 246
							— 3 114
						Error	132

In Column 8 the minus sign indicates that the water is flowing out of, and the plus sign that it is flowing into, the Upper Bay.

TABLE 9.—Rates of Flow, in Millions of Cubic Feet per Lunar Hour.

Lunar Hours.	The Narrows.		Hudson River.		East River.		Kill van Kull.		Water in Upper Bay.	
	Into Upper Bay.	Out of Upper Bay.	Into Upper Bay.	Out of Upper Bay.	Into Upper Bay.	Out of Upper Bay.	Into Upper Bay.	Out of Upper Bay.	Decreasing.	Increasing.
I	.....	3 114	1 672	.....	1 129	.....	175	.....	270	.....
II	.....	2 450	1 820	.....	888	.....	.....	43	.....	69
III	.....	1 130	1 480	.....	410	.....	.....	240	.....	390
IV	441	.....	744	.....	.....	176	.....	372	.....	606
V	1 775	.....	.....	162	.....	707	.....	406	.....	659
VI	2 634	.....	.....	909	.....	1 048	.....	330	.....	536
VII	2 786	.....	.....	1 412	.....	1 109	.....	166	.....	270
VIII	2 192	.....	.....	1 537	.....	873	45	.....	69	.....
IX	1 011	.....	.....	1 251	.....	402	253	.....	390	.....
X	.....	493	.....	629	179	.....	393	.....	606	.....
XI	.....	1 984	191	.....	719	.....	428	.....	659	.....
XII	.....	2 944	1 076	.....	1 067	.....	348	.....	536	.....

Division of Sea- and Land-Water.

The salinity of the water in different parts of the harbor was observed by the Metropolitan Sewerage Commission from September, 1908, until December, 1909, inclusive. Eleven observation stations were established, as shown in Fig. 1. At each station observers were trained during the last months of 1908, and a complete record was made for every day of the year, 1909. Samples of water were taken each day at three different times, usually at 8.00 A. M., 12.00 Noon, and 4.00 P. M.

The term "land-water," as used in this investigation, represents any water between fresh-water and water containing approximately 300 parts of chlorine per 1 000 000 parts. Between these limits the observers would record the water as being 100% of land-water. The standard for "sea-water" was a water containing a salinity of 18 000 parts of chlorine per 1 000 000 parts, assuming that the chlorides represent 88.6% of the total specific gravity of sea-water. Under this condition, the salinometer reading would be 1.025 when corrected to a standard temperature of 60° Fahr.

The instruments were all calibrated, and, as a further check, their readings were compared with results obtained by chemical analyses. The indicated chlorine and the actual chlorine, as found by analysis, agreed very closely.*

The quantity of land-water discharged into the harbor varies according to the season, being greatest during April and least during August. When the discharge of land-water is at a maximum, the salt-water of the sea is driven down the Hudson, so that the river at Tarrytown is practically fresh. When the discharge of land-water is at a minimum, the salt-water works its way up the Hudson as far as Poughkeepsie.

If no land-water entered the harbor, the water therein would be as salty as that of the sea. As the record shows that the harbor water is salty, but less salty than sea-water, it is evident that the former is a mixture of sea-water and land-water. It was considered that these salinometer records could be used in finding the proportions of sea- and land-water in the harbor waters, and a study was made of them for this purpose.

About 13 000 observations for salinity were made in the harbor during 1909. Additional observations were made at different depths

* Report, Metropolitan Sewerage Commission, April 30th, 1910, p. 519.

below the surface. These indicated that, usually, the water was slightly more salty as the depth from the surface increased, that the difference was small, and that the difference in salinity at the surface and at various depths depended on the currents at the place of observation. If the currents were irregular in character, the difference in salinity was very small.

These observations were tabulated. The monthly averages of land-water are given in Table 10 for the different stations. In this table, the figures in the columns headed "All" are the averages of all the observations taken during a month. The figures in the column headed "Out" and "In" are the averages of all the observations taken during inflowing or outflowing currents. Some observations were taken when the direction of the current was doubtful, as at slack water, and these are not included under either "In" or "Out," but under "All."

A study was made of the records of the Weather Bureau, U. S. Department of Agriculture, so as to compare the rainfall of 1909 with the normal. The monthly and yearly precipitation at Albany, Carmel, Wappinger Falls, West Point, and New York, were averaged. The results of these averages are given in the last line of Table 10, and show the percentage of precipitation above or below the normal for each month of 1909.

The volumes of water flowing on ebb and flood currents, for different parts of the harbor, are stated under the heading, "Volumes of Tidal Discharges." From these data the volumes flowing at intermediate places were estimated (see Appendix B). These volumes are given in Table 11.

Additional information regarding specific gravities and temperatures of the water is given in Appendix D, as recorded by the salinity observations made by the Metropolitan Sewerage Commission.

The Upper Bay and Hudson River.—A longitudinal section was taken from The Narrows, through the Upper Bay and Hudson River as far as Tarrytown, the distance between the places being laid off as abscissas in Fig. 39. At the places named, the mean tidal flows of ebb tide, in cubic feet, were laid off as ordinates, and a smooth curve was drawn through these points.

This mean quantity of water flowing on ebb tide was then divided into land- and sea-water, using the salinometer percentages given in

TABLE 10.—Summary of Salinometer Records for 1909.

Percentage of Land-Water at the Surface. Corrected for Temperature 60 degrees.

	Jan.			Feb.			Mar.			Apr.			May.			June.			July.		
	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.
Ambrose Channel	1.3	....	....	6.5	....	....	10.2	....	....	10.8	....	....	15.6	....	....	13.0	....	....	6.9	....	....
West Bank	19.3	19.7	18.3	34.2	35.8	33.4	35.0	34.5	34.9	45.1	44.3	45.6	42.2	41.7	41.6	27.8	28.5	27.3	17.3	17.1	17.5
Fort Wadsworth	29.0	28.2	30.0	50.7	50.8	52.4	50.5	49.9	51.0	64.9	59.8	67.9	59.1	56.0	61.0	38.4	38.0	39.1	25.5	24.3	28.2
Robbins Reef	24.9	23.3	24.6	42.6	41.8	42.8	43.2	43.3	44.4	56.1	55.6	55.8	52.4	52.5	51.6	34.0	33.6	33.8	21.5	21.1	21.6
Governors Island	30.9	29.4	33.7	40.5	39.0	43.4	49.3	49.1	49.9	64.0	60.1	67.2	59.1	56.6	61.0	39.5	38.7	38.9	25.3	24.6	27.5
Blackwells Island	26.5	26.5	26.1	35.9	36.9	34.6	37.8	38.0	37.6	45.4	45.3	44.9	42.3	41.7	42.9	33.7	34.0	33.4	27.7	26.8	28.2
Great Beds	31.7	33.1	29.2	47.1	49.5	44.1	46.5	48.6	44.5	58.2	59.4	57.0	53.6	55.5	50.7	37.6	38.9	36.1	28.7	29.3	28.0
Passaic Light	65.8	63.8	67.3	82.2	81.0	84.0	83.4	83.5	83.3	90.6	90.3	90.9	86.3	85.7	86.8	67.2	66.3	67.8	51.0	50.0	51.8
Fort Washington Point	62.2	60.7	63.2	83.3	83.2	83.7	84.0	83.9	84.0	94.7	94.7	95.1	90.6	90.7	90.5	74.5	74.4	74.7	56.0	56.0	55.7
Tarrytown	84.6	84.8	84.5	97.2	97.4	97.0	98.6	98.6	98.7	100.0	100.0	100.0	99.9	99.8	99.9	93.1	93.4	92.8	80.2	80.4	80.0
Throgs Neck	15.9	15.8	16.0	19.3	19.8	18.7	20.8	21.2	20.7	23.8	23.5	24.2	24.3	24.0	24.5	21.3	21.4	21.2	18.1	18.2	18.0
Rainfall, percentage above or below normal.*	— 6%			+ 32%			— 23%			+ 61%			— 36%			+ 13%			— 53%		

	Aug.			Sept.			Oct.			Nov.			Dec.			Averages for Year.		
	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.
Ambrose Channel	7.2	....	....	6.8	....	....	6.0	....	....	4.6	....	....	4.6	....	....	7.6	.....	.....
West Bank	18.4	17.5	19.4	16.2	16.6	16.0	15.7	15.4	16.0	15.2	14.7	15.7	17.5	17.4	17.9	25.2	24.8	24.1
Fort Wadsworth	29.5	28.7	30.2	28.3	24.8	32.3	22.2	21.5	25.5	22.7	22.2	23.0	25.3	23.6	27.3	37.2	34.8	37.5
Robbins Reef	21.4	21.6	21.3	18.3	18.3	18.6	19.6	19.3	20.4	19.7	19.7	19.5	22.3	21.9	22.8	31.2	30.4	30.4
Governors Island	24.9	23.8	26.0	22.2	19.7	24.9	24.7	22.8	27.0	25.1	24.1	26.4	30.0	28.1	32.5	36.4	34.6	37.0
Blackwells Island	25.5	26.5	24.4	24.1	24.3	23.8	25.6	25.2	26.2	24.2	24.5	23.8	25.7	25.6	25.8	31.9	31.9	31.9
Great Beds	29.6	31.4	27.1	26.2	26.4	25.9	23.9	24.5	23.4	24.9	25.0	24.9	23.6	23.2	23.9	35.8	37.2	34.5
Passaic Light	52.6	52.2	53.6	47.5	47.3	48.3	46.4	46.2	46.1	45.3	44.6	45.8	57.6	56.0	59.0	64.2	63.8	64.9
Fort Washington Point	51.8	52.4	50.6	48.8	49.0	48.3	50.4	49.5	50.8	49.4	49.7	48.4	53.7	53.0	54.4	66.3	66.3	65.1
Tarrytown	74.4	75.2	73.7	73.0	73.5	72.6	73.8	74.4	73.4	73.4	73.5	73.3	76.4	76.5	76.3	85.3	85.7	84.9
Throgs Neck	17.3	17.7	17.0	17.1	16.9	17.4	15.5	15.0	15.9	13.2	13.3	13.2	11.6	11.4	11.7	18.1	18.2	18.1
Rainfall, percentage above or below normal. *	+ 14%			— 11%			— 74%			— 47%			+ 9%			— 12%		

* Average of precipitation at Albany, Carmel, New York City, Wappinger Falls, and West Point, from climatological Report, U. S. Weather Bureau.

Throgs Neck : West currents = In.

" All " means average of Ebb, Flood and Slack Water observations.

" Out " means average of Ebb observations.

" In " means average of Flood observations.

Table 10, and a smooth curve was drawn through these points, allowance being made if the precipitation was above or below the annual normal. This gave the middle line in the diagram, dividing land-water from sea-water.

TABLE 11.—Volumes Flowing on Ebb and Flood Currents, Harbor of New York, in Millions of Cubic Feet.

Part of Harbor.	Yearly Means.	
	Ebb.	Flood.
The Narrows	12 041	10 779
Hudson River, off The Battery	7 430	6 343
" " " 39th Street	6 990	5 903
" " " Fort Washington Point	6 230	5 143
" " " Tarrytown	3 980	2 893
East River, off Brooklyn Bridge	4 370	4 290
" " " East 11th Street, Manhattan	4 339	4 259
" " " " 31st Street, "	4 298	4 218
" " " " 81st Street, "	3 943	3 863
" " " Old Ferry Point	3 712	3 632
Newark Bay	1 972	1 866

The land-water was then divided into "land-water which will return" and "land-water which will not return." This was done by plotting to scale at the different places the mean discharge of land-water for 12 lunar hours above the middle line. The remainder of the land-water is "land-water which will return." The discharge of land-water is the same as that given in Table 5. The land-water is held back by flood tide, so that the discharge for 12 hours has to pass out during ebb tide. The principle on which this division was made is, that if the density of the water in two successive ebb tides is to remain constant, the same quantity of land-water must flow out and not return on each ebb tide.

The sea-water was divided into "sea-water which will return" and "sea-water which will not return" by making the "sea-water which will not return" the same proportion of the total sea-water, as the "land-water which will not return" is of the total land-water. This is so because the land- and sea-waters are mixed, and the quantity of "land-water which will not return" will take with it its proportionate quantity of sea-water.

The division of sea- and land-water in a flood tide, when the yearly mean quantity of land-water is flowing, is shown in Fig. 40. This diagram was constructed by laying off the tidal flow of the mean flood,

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER.
WHEN AVERAGE LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

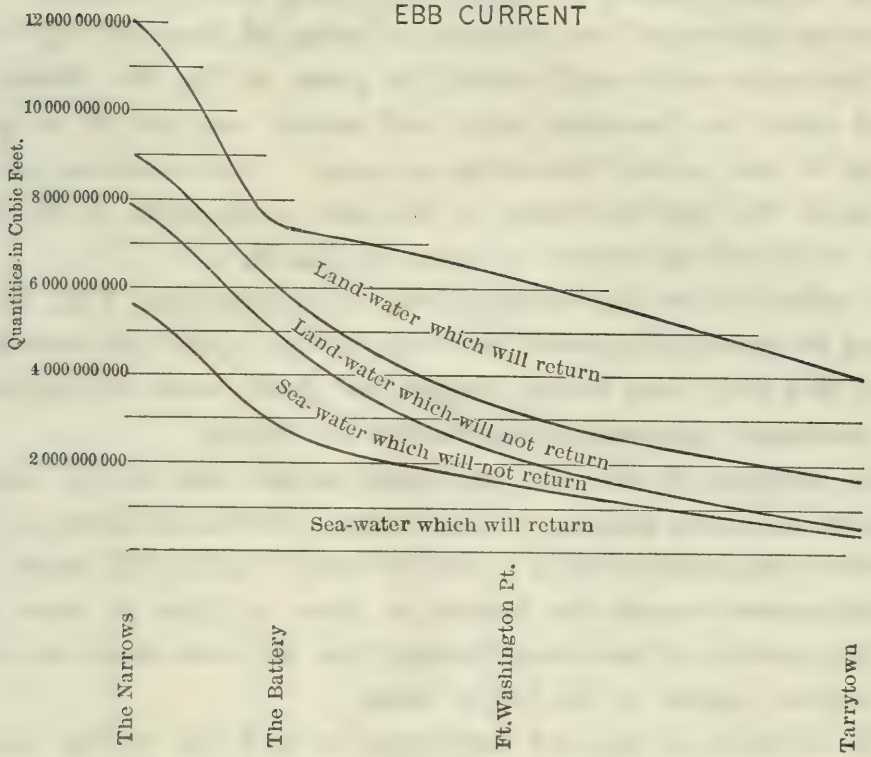


FIG. 39.

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER.
WHEN AVERAGE LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

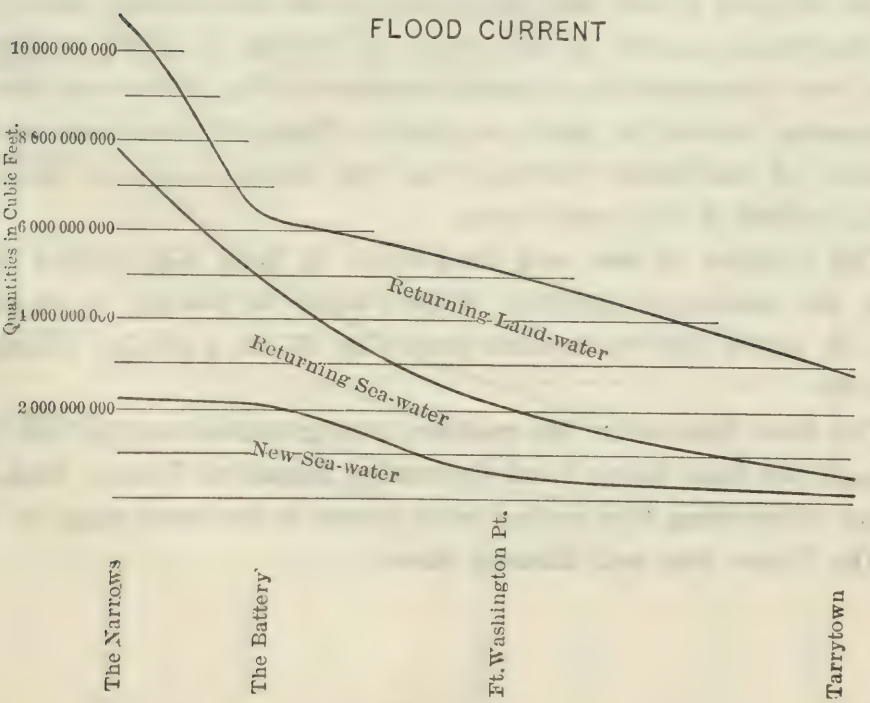


FIG. 40.

in cubic feet, as ordinates, and drawing a smooth curve through the points. This result is the same as subtracting from the top curve of Fig. 39, the quantity of "land-water which will not return." The "returning land-water" was obtained by laying off, from this top curve, the "land-water which will return," as given in Fig. 39. Below the second curve, the "sea-water which will return" was laid off, as given in Fig. 39, and marked "returning sea-water." The remaining part of the figure, the "new sea-water," is the same in quantity as the "sea-water which will not return," as given in Fig. 39.

In order to show the variation from mean conditions, Figs. 41, 42, 43, and 44 were constructed to show the division of sea- and land-water in ebb and flood tides during August and April, when the minimum and maximum quantities of land-water are flowing.

The division of sea- and land-water in ebb tide during August, when the minimum quantity of land-water is flowing, is shown in Fig. 41, which was constructed in a similar manner to Fig. 39, except that the salinometer records for August, as given in Table 10, were used, and the quantity of land-water flowing from the water-sheds was taken for August, instead of the yearly mean.

The division of sea- and land-water in flood tide during August, when the minimum quantity of land-water is flowing, is shown in Fig. 42, which was constructed from Fig. 41, in a similar manner to Fig. 40.

The division of sea- and land-water in ebb tide during April, when the maximum quantity of land-water is flowing, is shown in Fig. 43, which was constructed in a similar manner to Fig. 39, except that the salinometer records for April, as given in Table 10, were used, and the quantity of land-water flowing from the water-sheds was taken for April, instead of the yearly mean.

The division of sea- and land-water in flood tide during April, when the maximum quantity of land-water is flowing, is shown in Fig. 44, which was constructed from Fig. 43, in a similar manner to Fig. 40.

The East River.—In like manner, a longitudinal section was taken through the East River from Governors Island to Throgs Neck, and figures illustrating this section were drawn to the same scale as those for the Upper Bay and Hudson River.

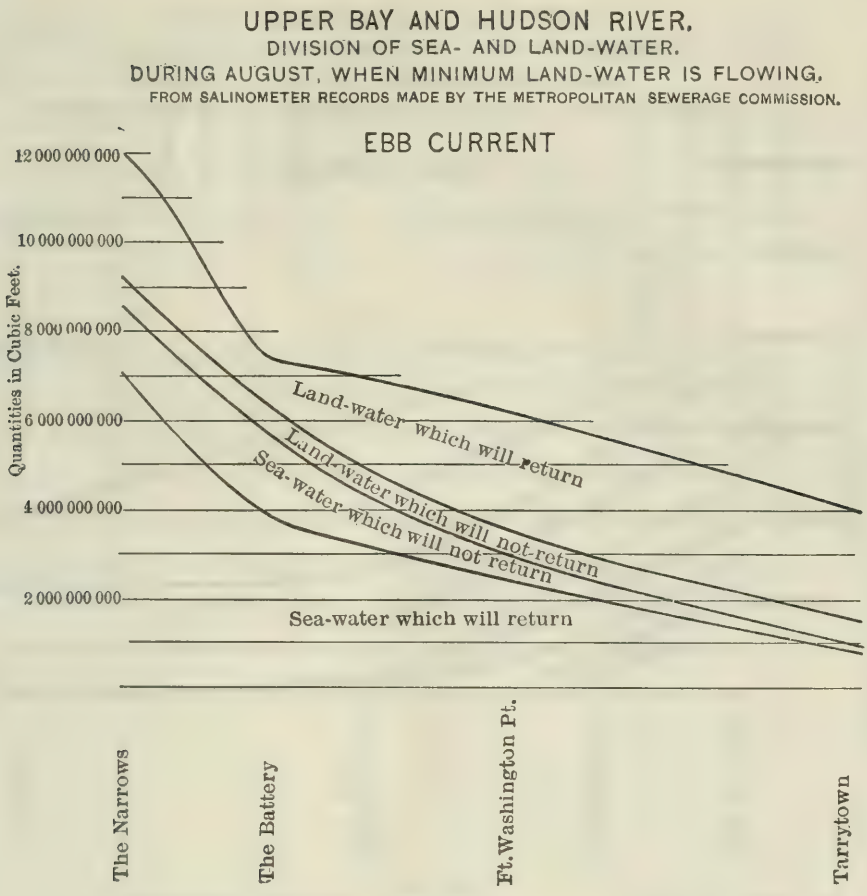

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER.
DURING AUGUST, WHEN MINIMUM LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.
EBB CURRENT
12 000 000 000
10 000 000 000
8 000 000 000
6 000 000 000
4 000 000 000
2 000 000 000
Quantities in Cubic Feet.
Land-water which will return
Land-water which will not return
Sea-water which will not return
Sea-water which will return
The Narrows
The Battery
Ft. Washington Pt.
Tarrytown

FIG. 41.

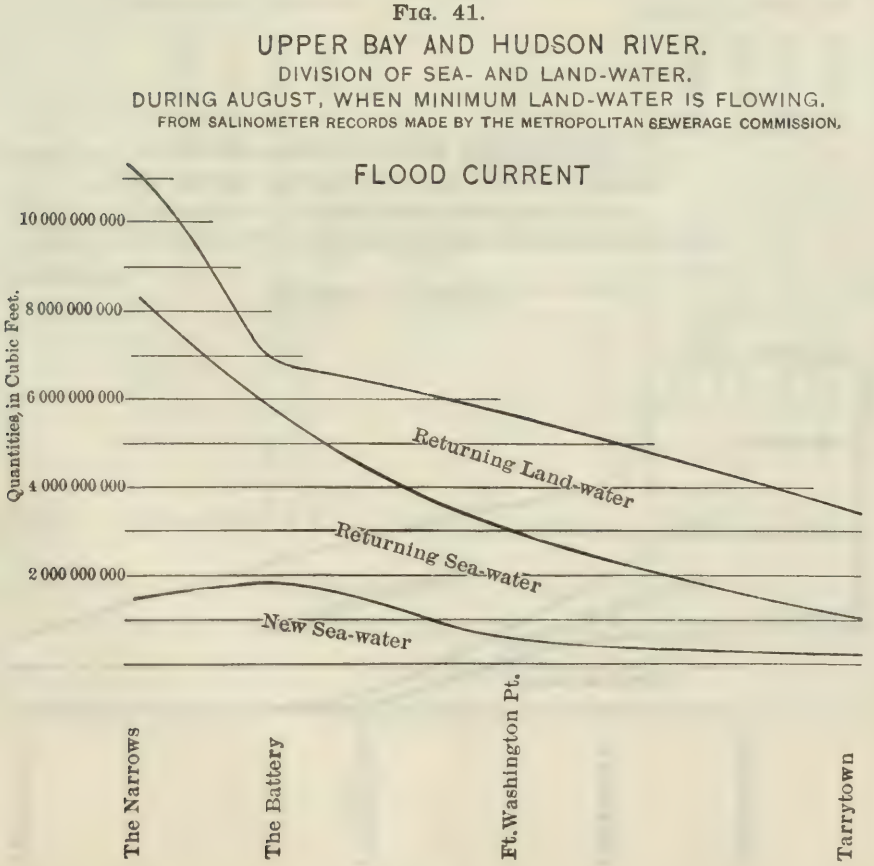

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER.
DURING AUGUST, WHEN MINIMUM LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.
FLOOD CURRENT
10 000 000 000
8 000 000 000
6 000 000 000
4 000 000 000
2 000 000 000
Quantities in Cubic Feet.
Returning Land-water
Returning Sea-water
New Sea-water
The Narrows
The Battery
Ft. Washington Pt.
Tarrytown

FIG. 42.

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER DURING APRIL,
WHEN MAXIMUM LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

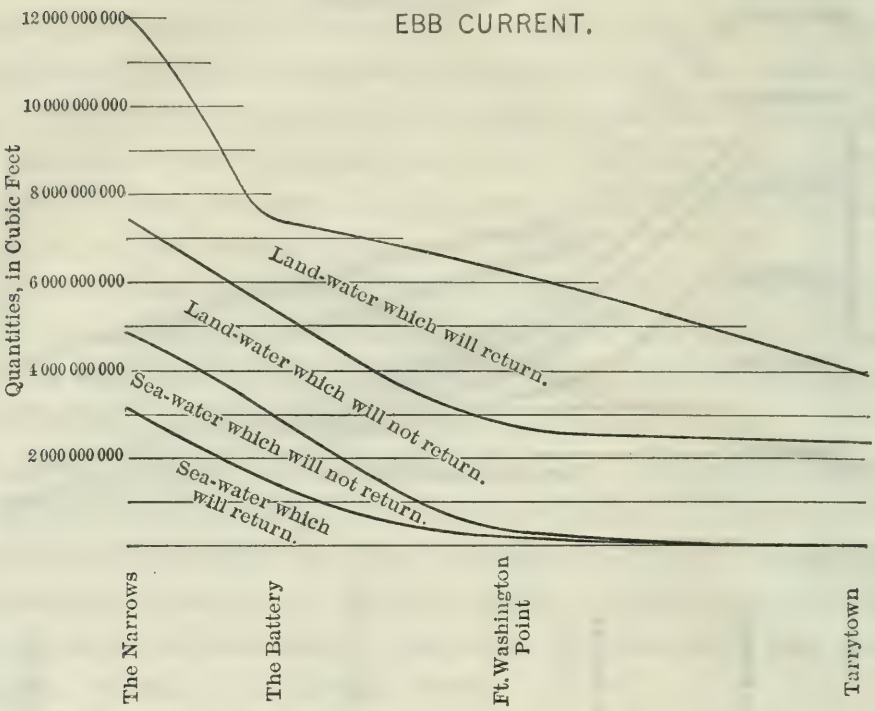


FIG. 43.

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER DURING APRIL,
WHEN MAXIMUM LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

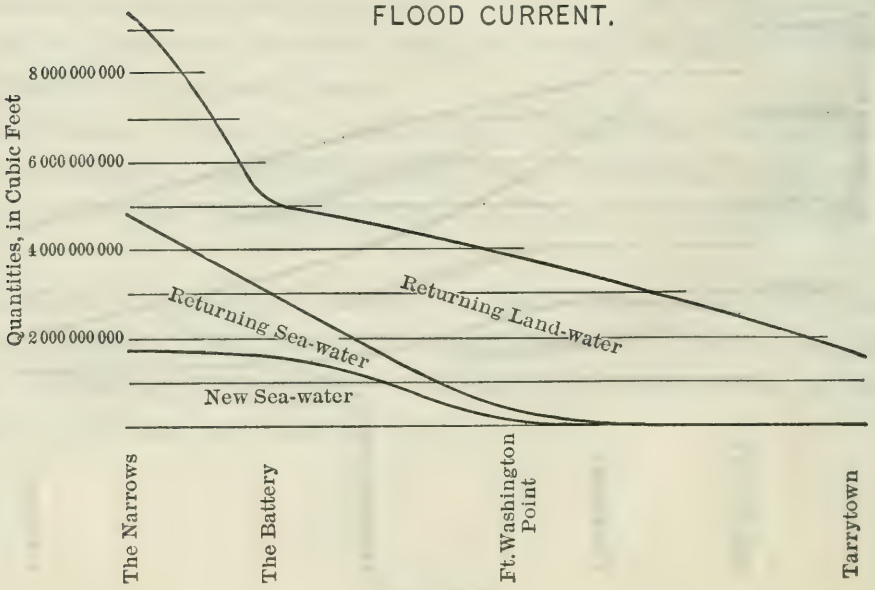


FIG. 44.

EAST RIVER.

DIVISION OF SEA- AND LAND-WATER WHEN AVERAGE LAND-WATER IS FLOWING.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

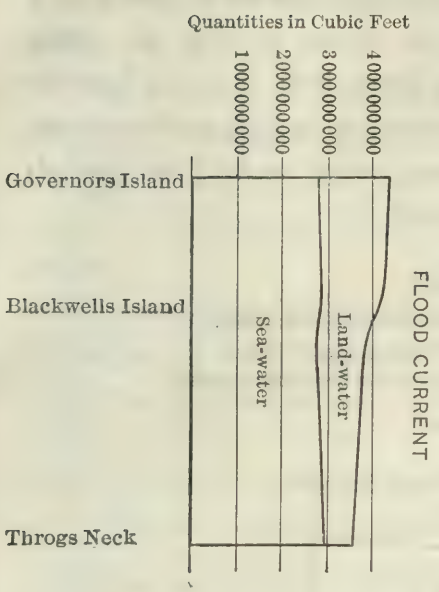


FIG. 45.

EAST RIVER.

DIVISION OF SEA- AND LAND-WATER DURING AUGUST, WHEN MINIMUM LAND-WATER IS FLOWING.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

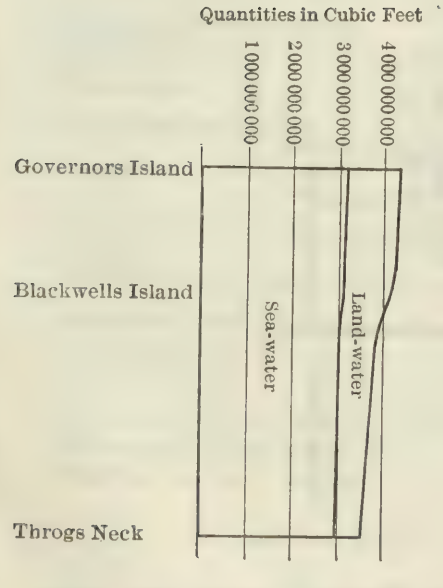


FIG. 47.

The division of sea- and land-water in ebb tide when the yearly mean quantity of land-water is flowing, is shown in Fig. 45. This diagram was constructed by laying off the mean flow, in cubic feet per tide, as ordinates, and dividing these ordinates according to the salinometer records, as given in Table 10, allowance being made if the precipitation was above or below the annual normal.

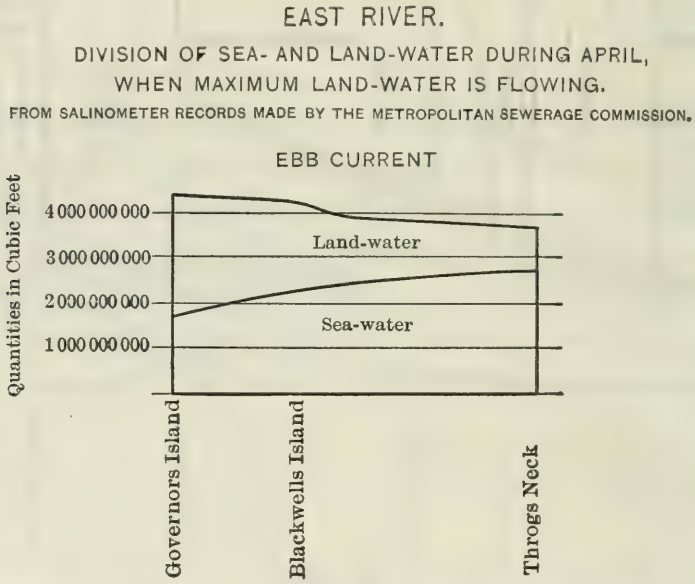


FIG. 49.

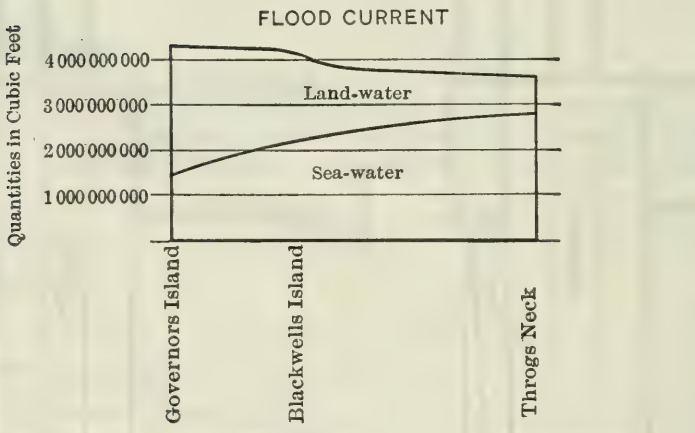


FIG. 50.

The division of sea- and land-water in flood tide when the yearly mean quantity of land-water is flowing, is shown in Fig. 46.

The division of sea- and land-water in ebb tide during August, when the minimum quantity of land-water is flowing, is shown in

Fig. 47. The division of sea- and land-water in flood tide during August, when the minimum quantity of land-water is flowing, is shown in Fig. 48.

The division of sea- and land-water in ebb tide during April, when the maximum quantity of land-water is flowing, is shown in Fig. 49. The division of sea- and land-water in flood tide during April, when the maximum quantity of land-water is flowing, is shown in Fig. 50.

The results shown graphically in Figs. 39 to 50, inclusive, are given in Tables 12 to 17, inclusive. These tables also give the percentages which the division of sea- and land-water are of the total tidal flows.

TABLE 12.—Division of Sea- and Land-Water in Ebb Current when Yearly Average Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Ebb Current.

Part of Harbor.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
*Upper Bay and Hudson River.					
The Narrows	3 010	1 180	5 640	2 210	12 040
	25.0%	9.8%	46.8%	18.4%	100%
The Battery	1 480	1 090	2 800	2 060	7 430
	19.9%	14.7%	37.7%	27.7%	100%
Fort Washington Point	3 040	1 090	1 550	550	6 230
	48.8%	17.5%	24.9%	8.8%	100%
Tarrytown	2 320	1 090	390	180	3 980
	58.3%	27.4%	9.8%	4.5%	100%
† East River.					
Governors Island	1 520		2 870		4 390
	34.6%		65.4%		100%
Blackwells Island	1 320		2 830		4 150
	31.9%		68.1%		100%
Throgs Neck	660		3 000		3 660
	18.1%		81.9%		100%

* Shown graphically on Fig. 39.
† Shown graphically on Fig. 45.

Division of Sea- and Land-Water by Months.

A study was made of the division of sea- and land-water in both ebb and flood tides, to show the monthly variations at different places in the harbor.

TABLE 13.—Division of Sea- and Land-Water in Flood Current when Yearly Average Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Flood Current.

Part of Harbor.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
* Upper Bay and Hudson River.				
The Narrows	3 010	5 560‡	2 210	10 780
	27.9%	51.6%	20.5%	100%
The Battery	1 480	2 800	2 060	6 340
	23.3%	44.2%	32.5%	100%
Fort Washington Point	3 040	1 550	550	5 140
	59.1%	30.2%	10.7%	100%
Tarrytown	2 320	390	180	2 890
	80.3%	13.5%	6.2%	100%
† East River.				
Governors Island	1 590	2 720		4 310
	37.0%	63.0%		100%
Blackwells Island	1 300	2 770		4 070
	31.9%	68.1%		100%
Throgs Neck	650	2 930		3 580
	18.2%	81.8%		100%

* Shown graphically on Fig. 40. † Shown graphically on Fig. 46.

‡ This figure is 80 000 000 cu. ft. less than the "sea-water which will return," as given in Table 12, because this is the mean resultant flow through the East River.

TABLE 14.—Division of Sea- and Land-Water in Ebb Current, During August, when Minimum Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Ebb Current.

Part of Harbor.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
* Upper Bay and Hudson River.					
The Narrows	2 830	620	7 050	1 540	12 040
	23.5%	5.2%	58.5%	12.8%	100%
The Battery	1 200	570	3 830	1 830	7 430
	16.1%	7.7%	51.6%	24.6%	100%
Fort Washington Point	2 700	570	2 450	510	6 230
	43.3%	9.1%	39.3%	8.3%	100%
Tarrytown	2 420	570	800	190	3 980
	60.9%	14.3%	20.1%	4.7%	100%
† East River,					
Governors Island	1 040		3 350		4 390
	23.8%		76.2%		100%
Blackwells Island	1 100		3 050		4 150
	26.5%		73.5%		100%
Throgs Neck	620		3 040		3 660
	17.0%		83.0%		100%

* Shown graphically on Fig. 41. † Shown graphically on Fig. 47.

TABLE 15.—Division of Sea- and Land-Water in Flood Current, During August, when Minimum Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Flood Current.

Part of Harbor.	Land-Water returning.	Sea-Water.		Total flood flow.
		Returning.	New.	
*Upper Bay and Hudson River.				
The Narrows	2 830	6 970‡	1 540	11 340
	24.9%	61.5%	13.6%	100%
The Battery	1 200	3 830	1 830	6 860
	17.5%	55.8%	26.7%	100%
Fort Washington Point	2 700	2 450	510	5 660
	47.7%	43.3%	9.0%	100%
Tarrytown	2 420	800	190	3 410
	71.0%	23.4%	5.6%	100%
†East River.				
Governors Island	1 120	3 190		4 310
	26.0%	74.0%		100%
Blackwells Island	990	3 080		4 070
	24.4%	75.6%		100%
Throgs Neck	630	2 950		3 580
	17.7%	82.3%		100%

* Shown graphically on Fig. 42. † Shown graphically on Fig. 48.

‡ This figure is 80 000 000 cu. ft. less than the "sea-water which will return," as given in Table 14, because this is the mean resultant flow through the East River.

TABLE 16.—Division of Sea- and Land-Water in Ebb Current, During April, when Maximum Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Ebb Current.

Part of Harbor.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
*Upper Bay and Hudson River.					
The Narrows	4 610	2 590	3 100	1 740	12 040
	38.3%	21.5%	25.7%	14.5%	100%
The Battery	2 090	2 380	1 380	1 580	7 430
	28.1%	32.0%	18.6%	21.3%	100%
Fort Washington Point	3 520	2 380	200	130	6 230
	56.5%	38.2%	3.2%	2.1%	100%
Tarrytown	1 600	2 380	....	....	3 980
	40.2%	59.8%	....	....	100%
†East River.					
Governors Island	2 640		1 750		4 390
	60.1%		39.9%		100%
Blackwells Island	1 880		2 270		4 150
	45.3%		54.7%		100%
Throgs Neck	890		2 770		3 660
	24.2%		75.8%		100%

* Shown graphically on Fig. 43. † Shown graphically on Fig. 49.

TABLE 17.—Division of Sea- and Land-Water in Flood Current, During April, when Maximum Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Flood Current.

Part of Harbor.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
*Upper Bay and Hudson River.				
The Narrows	4 610	3 020‡	1 740	9 370
	49.2%	32.2%	18.6%	100%
The Battery	2 090	1 380	1 580	5 050
	41.4%	27.3%	31.3%	100%
Fort Washington Point	3 520	200	130	3 850
	91.4%	5.2%	3.4%	100%
Tarrytown	1 600	.....	.....	1 600
	100%	.....	.....	100%
†East River.				
Governors Island	2 900	1 410		4 310
	67.2%	32.8%		100%
Blackwells Island	1 830	2 240		4 070
	44.9%	55.1%		100%
Throgs Neck	840	2 740		3 580
	23.5%	76.5%		100%

*Shown graphically on Fig. 44.

†Shown graphically on Fig. 50.

‡This figure is 80 000 000 cu. ft. less than the "sea water which will return," as given in Table 16, because this is the mean resultant flow through the East River.

The Narrows.—The division of sea- and land-water in ebb tides by months is shown in Fig. 51. This diagram was constructed by laying off monthly divisions as abscissas, and the mean quantity of water flowing in ebb tide as ordinates. The curves dividing the "land-water which will return" and the "land-water which will not return" and the "sea-water which will return" and the "sea-water which will not return," were drawn in a similar manner to the curves shown on Fig. 39; except that the salinometer records were taken for each month from Table 10, instead of the mean for the year, and also the quantities of land-water flowing were taken for the different months, instead of the yearly mean.

The division of sea- and land-water in flood tides by months is shown in Fig. 52. This diagram was constructed by deducting from Fig. 51 the quantity of "land-water which will not return." This gives the upper curve of the diagram. The returning land-water was plotted by laying off, from this upper curve, the quantity of "land-water which will return," as given in Fig. 51. In a similar manner, the "returning

THE NARROWS.

DIVISION OF SEA- AND LAND-WATER BY MONTHS.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

EBB CURRENT.

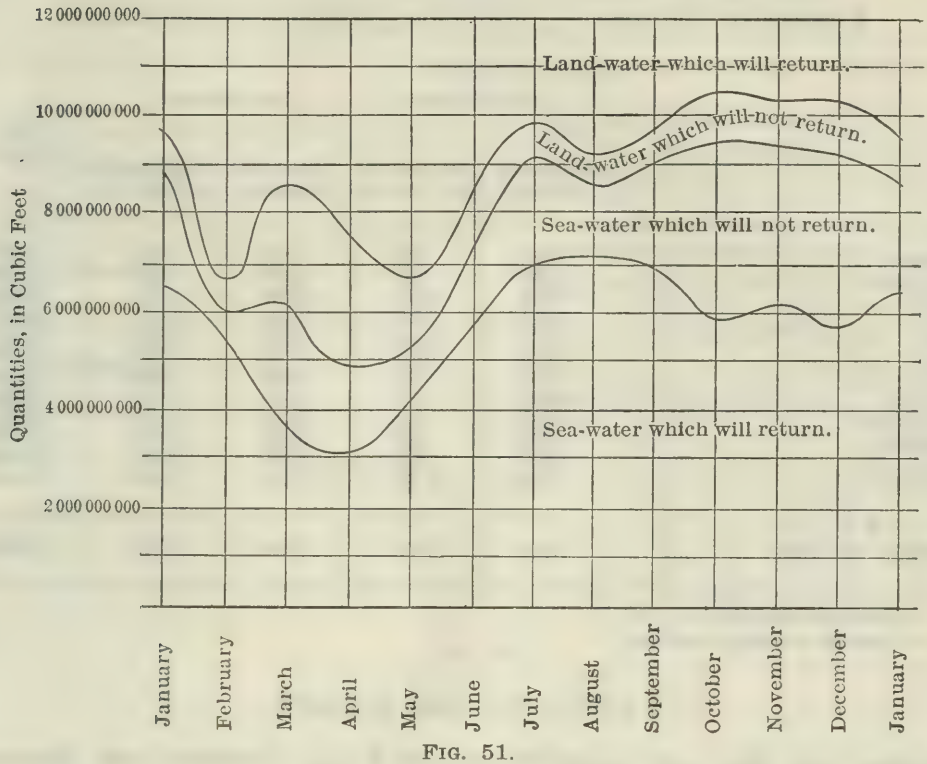


FIG. 51.

FLOOD CURRENT.

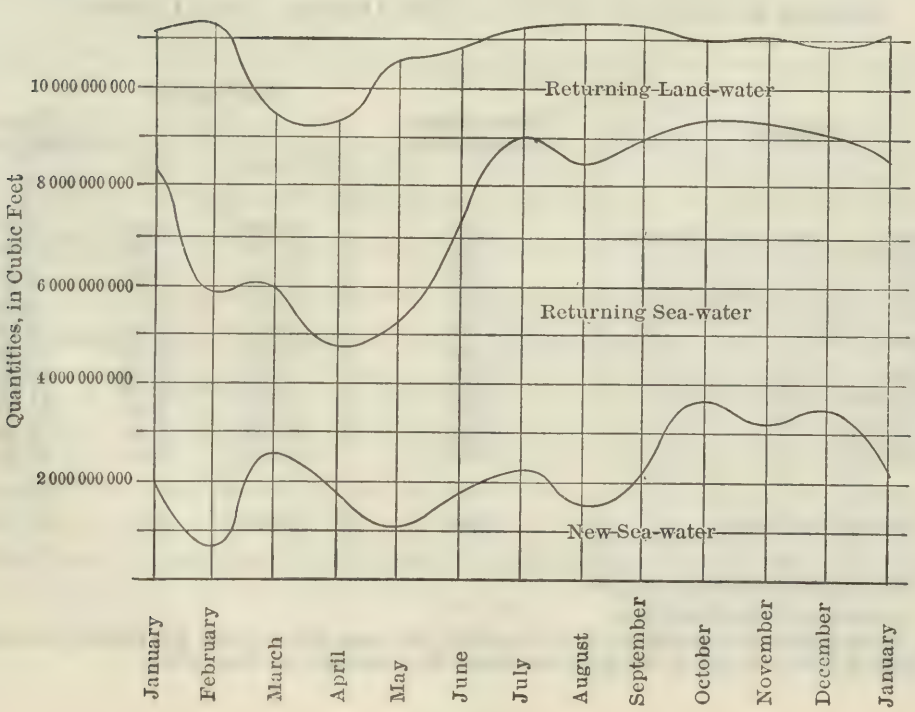


FIG. 52.

TABLE 18.—The Narrows.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
January	2 520	880	6 400	2 240	12 040
February	5 430	690	5 250	670	12 040
March	3 510	2 500	3 520	2 510	12 040
April	4 610	2 590	3 100	1 740	12 040
May	5 400	1 340	4 250	1 050	12 040
June	3 480	1 090	5 690	1 780	12 040
July	2 230	700	6 930	2 180	12 040
August	2 840	620	7 060	1 520	12 040
September	2 280	710	6 900	2 150	12 040
October	1 600	990	5 840	3 610	12 040
November	1 760	910	6 180	3 190	12 040
December	1 770	1 070	5 730	3 470	12 040
Average for Year†	3 010	1 180	5 640	2 210	12 040

*Shown graphically on Fig. 51.

†Average of daily readings.

TABLE 19.—The Narrows.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
January	2 520	6 320‡	2 240	11 080
February	5 430	5 170	670	11 270
March	3 510	3 440	2 510	9 460
April	4 610	3 020	1 740	9 370
May	5 400	4 170	1 050	10 620
June	3 480	5 610	1 780	10 870
July	2 230	6 850	2 180	11 260
August	2 840	6 980	1 520	11 340
September	2 280	6 820	2 150	11 250
October	1 600	5 760	3 610	10 970
November	1 760	6 100	3 190	11 050
December	1 770	5 650	3 470	10 890
Average for Year†	3 010	5 560	2 210	10 780

*Shown graphically on Fig. 52.

†Average of daily readings.

‡These figures are 80 000 000 cu. ft. less than the "sea-water which will return," as given in Table 18, because this is the mean resultant flow through the East River.

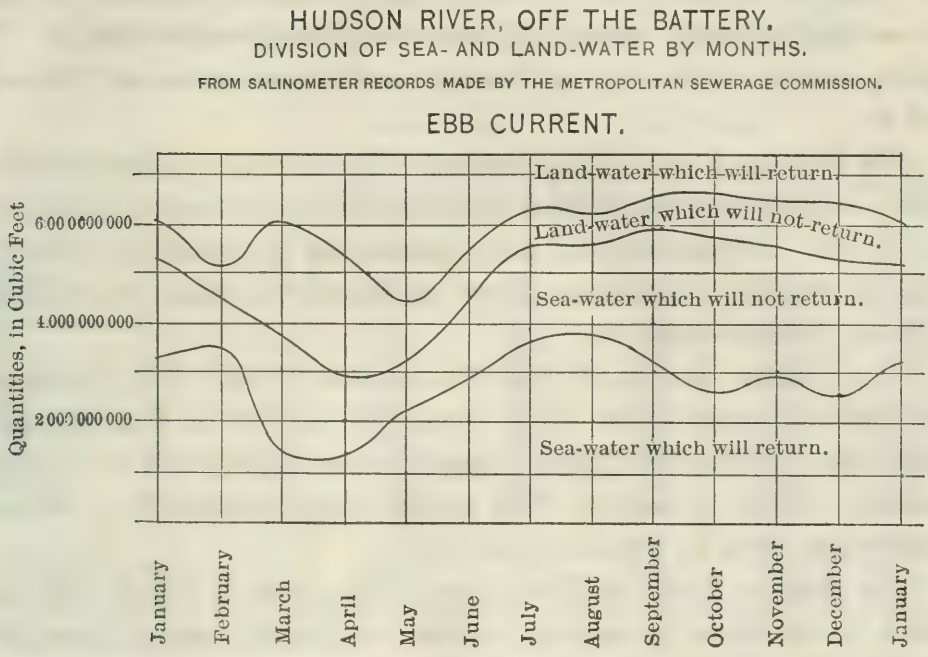

HUDSON RIVER, OFF THE BATTERY.
DIVISION OF SEA- AND LAND-WATER BY MONTHS.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.
EBB CURRENT.
Quantities, in Cubic Feet
6 000 000 000
4 000 000 000
2 000 000 000
Land-water which will return.
Land-water which will not return.
Sea-water which will not return.
Sea-water which will return.
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 53.

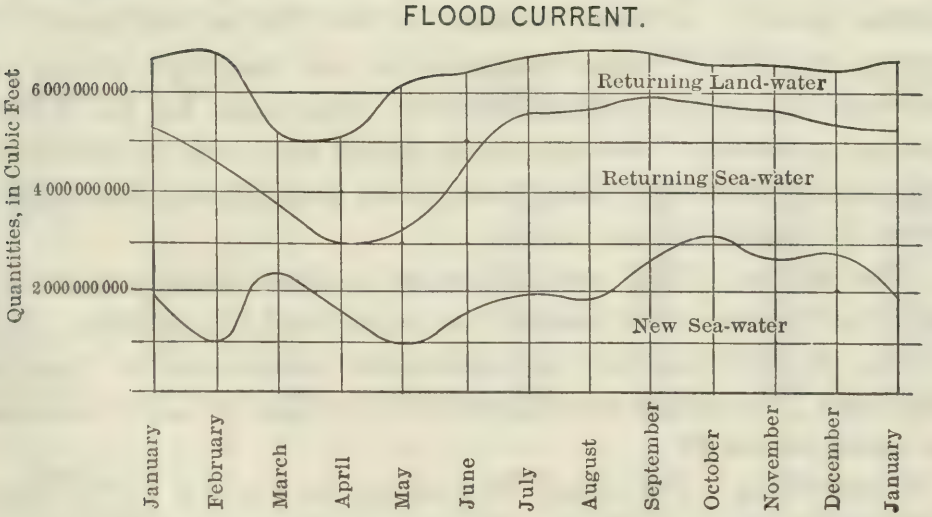

FLOOD CURRENT.
Quantities, in Cubic Feet
6 000 000 000
4 000 000 000
2 000 000 000
Returning Land-water
Returning Sea-water
New Sea-water
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 54.

sea-water" was laid off, and the difference (the quantity to complete the diagram) is, therefore, "new sea-water," which is the same in quantity as the "sea-water which will not return," as given in Fig. 51. The results shown graphically in Figs. 51 and 52 are given in Tables 18 and 19.

The Hudson River, off the Battery.—The division of sea- and land-water in ebb tides, by months, is given in Fig. 53, and in flood tides in Fig. 54. These diagrams were constructed in a similar manner to Figs. 51 and 52. The results shown graphically in Figs. 53 and 54 are given in Tables 20 and 21.

The Hudson River, off Fort Washington Point.—The division of sea- and land-water in ebb tides, by months, is given in Fig. 55, and in flood tides in Fig. 56. These diagrams were constructed in a similar manner to Figs. 51 and 52. The results shown graphically in Figs. 55 and 56 are given in Tables 22 and 23.

The Hudson River, off Tarrytown.—The division of sea- and land-water in ebb tides, by months, is given in Fig. 57, and in flood tides, in Fig. 58. These figures were constructed in a similar manner to Figs. 51 and 52. The results shown graphically in Figs. 57 and 58 are given in Tables 24 and 25.

The East River, off Blackwells Island.—The division of sea- and land-water in ebb tides, by months, is given in Fig. 59. This diagram was constructed by laying off monthly divisions as abscissas, and the average quantity of water flowing in ebb tide as ordinates. The average quantity of water flowing in ebb tide was then divided into land- and sea-water by the salinometer percentages for the different months, as given in Table 10, and a smooth curve was drawn through these points, allowance being made if the precipitation was above or below the normal for that month.

The division of sea- and land-water in flood tides, by months, is given in Fig. 60. This diagram was constructed in a similar manner to Fig. 59, excepting that the salinometer percentages for flood tide were used. The results shown graphically in Figs. 59 and 60 are given in Tables 26 and 27.

Newark Bay.—The figures illustrating the division of the waters in Newark Bay are not drawn to the same scale as the preceding ones. The scale was increased in order to show more clearly the division of the waters.

TABLE 20.—Hudson River, off the Battery.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
January	1 370	810	3 300	1 950	7 430
February	2 260	640	3 530	1 000	7 430
March	1 350	2 300	1 400	2 380	7 430
April	2 090	2 380	1 380	1 580	7 430
May	2 980	1 230	2 280	940	7 430
June	1 880	1 000	2 970	1 580	7 430
July	1 190	640	3 640	1 960	7 430
August	1 200	570	3 830	1 830	7 430
September	810	650	3 310	2 660	7 430
October	780	910	2 650	3 090	7 430
November	950	840	2 990	2 650	7 430
December	1 010	1 080	2 580	2 760	7 430
Average for Year†	1 480	1 090	2 800	2 060	7 430

*Shown graphically on Fig. 53.
† Average of daily readings.

TABLE 21.—Hudson River, off the Battery.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
January	1 370	3 300	1 950	6 620
February	2 260	3 530	1 000	6 790
March	1 350	1 400	2 380	5 130
April	2 090	1 380	1 580	5 050
May	2 980	2 280	940	6 200
June	1 880	2 970	1 580	6 430
July	1 190	3 640	1 960	6 790
August	1 200	3 830	1 830	6 860
September	810	3 310	2 660	6 780
October	780	2 650	3 090	6 520
November	950	2 990	2 650	6 590
December	1 010	2 580	2 760	6 350
Average for Year†	1 480	2 800	2 060	6 340

*Shown graphically on Fig. 54.
† Average of daily readings.

TABLE 22.—Hudson River, off Fort Washington Point.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
January	2 970	810	1 930	520	6 230
February	4 540	640	920	130	6 230
March	2 930	2 300	560	440	6 230
April	3 520	2 380	200	130	6 230
May	4 420	1 230	450	130	6 230
June	3 640	1 000	1 250	340	6 230
July	2 850	640	2 240	500	6 230
August	2 700	570	2 450	510	6 230
September	2 400	650	2 500	680	6 230
October	2 170	910	2 220	930	6 230
November	2 260	840	2 280	850	6 230
December	2 220	1 080	1 970	960	6 230
Average for Year.†	3 040	1 090	1 550	550	6 230

* Shown graphically on Fig. 55.

† Average of daily readings.

TABLE 23.—Hudson River, off Fort Washington Point.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
January	2 970	1 930	520	5 420
February	4 540	920	130	5 590
March	2 930	560	440	3 930
April	3 520	200	130	3 850
May	4 420	450	130	5 000
June	3 640	1 250	340	5 230
July	2 850	2 240	500	5 590
August	2 700	2 450	510	5 660
September	2 400	2 500	680	5 580
October	2 170	2 200	930	5 300
November	2 260	2 280	850	5 390
December	2 220	1 970	960	5 150
Average for Year†	3 040	1 550	550	5 140

*Shown graphically on Fig. 56.

† Average of daily readings.

TABLE 24.—Hudson River, off Tarrytown.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
January	2 570	810	460	140	3 980
February	3 240	640	80	20	3 980
March	1 620	2 300	25	35	3 980
April	1 600	2 380	....	....	3 980
May	2 740	1 230	7	3	3 980
June	2 720	1 000	190	70	3 980
July	2 560	640	620	160	3 980
August	2 420	570	800	190	3 980
September	2 280	650	820	230	3 980
October	2 050	910	710	310	3 980
November	2 090	840	750	300	3 980
December	1 970	1 080	600	330	3 980
Average for Year†	2 320	1 090	390	180	3 980

*Shown graphically on Fig. 57.
† Average of daily readings.

TABLE 25.—Hudson River, off Tarrytown.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
January	2 570	460	140	3 170
February	3 240	80	20	3 340
March	1 620	25	35	1 680
April	1 600	...	...	1 600
May	2 740	7	3	2 750
June	2 720	190	70	2 980
July	2 560	620	160	3 340
August	2 420	800	190	3 410
September	2 280	820	230	3 330
October	2 050	710	310	3 070
November	2 090	750	300	3 140
December	1 970	600	330	2 900
Average for Year †	2 320	390	180	2 890

*Shown graphically on Fig. 58.
†Average of daily readings.

TABLE 26.—East River, off Blackwells Island.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water.	Sea-Water.	Total ebb flow.
January	1 100	3 050	4 150
February	1 530	2 620	4 150
March	1 580	2 570	4 150
April	1 880	2 270	4 150
May	1 730	2 420	4 150
June	1 410	2 740	4 150
July	1 110	3 040	4 150
August	1 100	3 050	4 150
September	1 010	3 140	4 150
October	1 050	3 100	4 150
November	1 020	3 130	4 150
December	1 060	3 090	4 150
Average for Year.†	1 320	2 830	4 150

* Shown graphically on Fig. 59.

† Average of daily readings.

TABLE 27.—East River, off Blackwells Island.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water.	Sea-Water.	Total flood flow.
January	1 060	3 010	4 070
February	1 410	2 660	4 070
March	1 530	2 540	4 070
April	1 830	2 240	4 070
May	1 750	2 320	4 070
June	1 360	2 710	4 070
July	1 150	2 920	4 070
August	990	3 080	4 070
September	970	3 100	4 070
October	1 070	3 000	4 070
November	970	3 100	4 070
December	1 050	3 020	4 070
Average for Year†	1 300	2 770	4 070

* Shown graphically on Fig. 60.

† Average of daily readings.

The division of sea- and land-water in ebb tides, by months, is shown in Fig. 61, and in flood tides, in Fig. 62. These figures were constructed in a similar manner to Figs. 51 and 52. The results shown graphically in Figs. 61 and 62 are given in Tables 28 and 29.

HUDSON RIVER, OFF FT. WASHINGTON POINT.
DIVISION OF SEA- AND LAND-WATER BY MONTHS.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

EBB CURRENT.

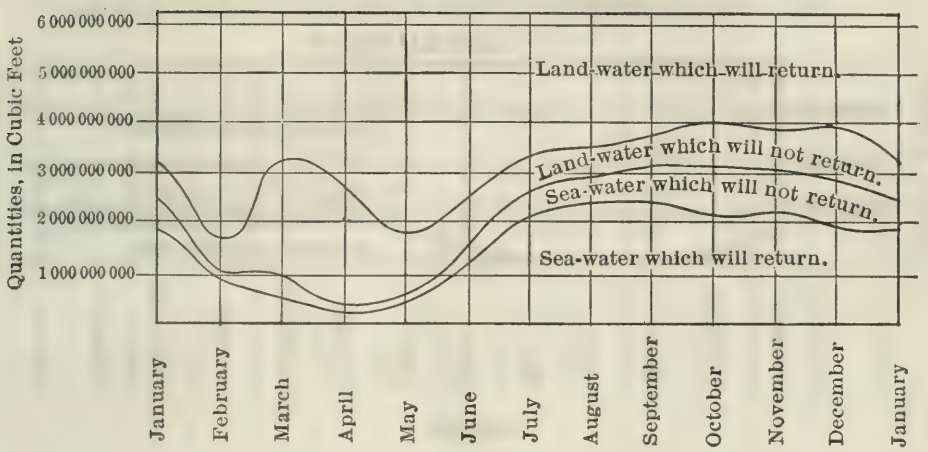


FIG. 55.

FLOOD CURRENT.

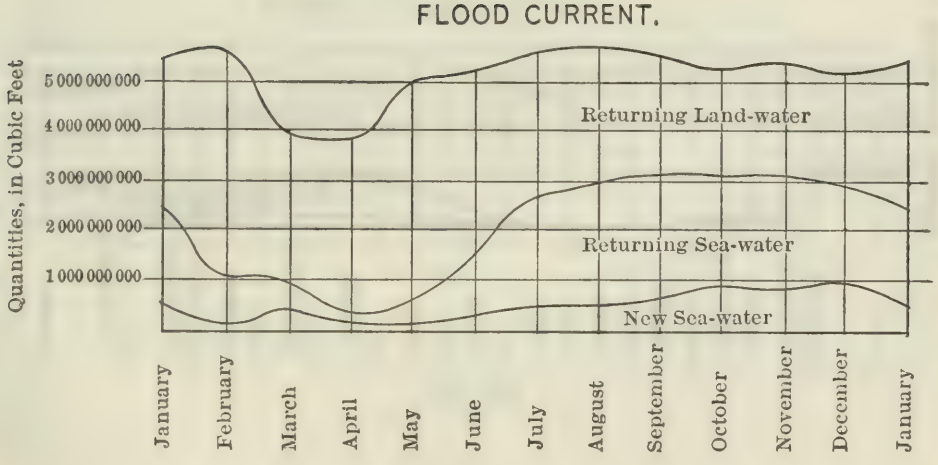


FIG. 56.

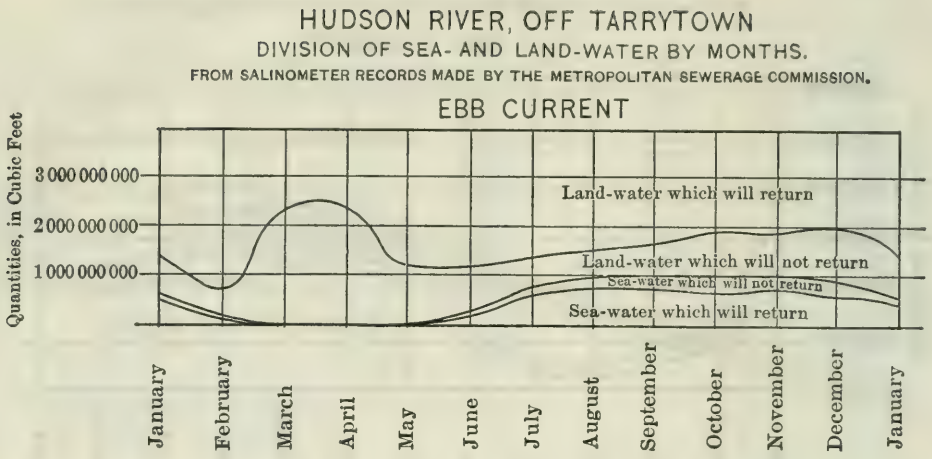

HUDSON RIVER, OFF TARRYTOWN
DIVISION OF SEA- AND LAND-WATER BY MONTHS.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.
EBB CURRENT
Quantities, in Cubic Feet
3 000 000 000
2 000 000 000
1 000 000 000
Land-water which will return
Land-water which will not return
Sea-water which will not return
Sea-water which will return
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 57.

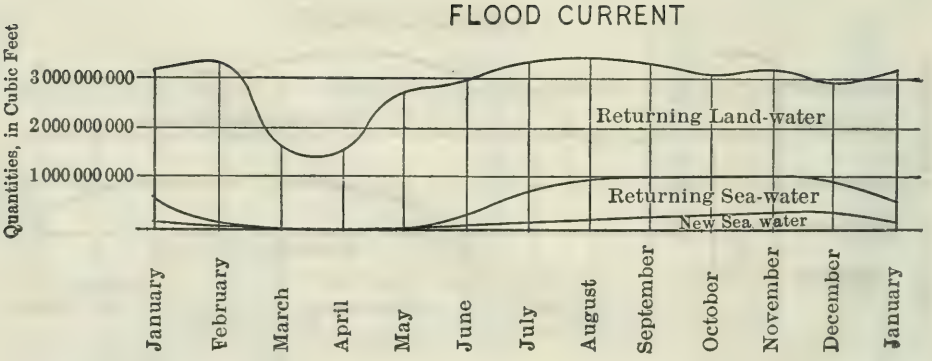

FLOOD CURRENT
Quantities, in Cubic Feet
3 000 000 000
2 000 000 000
1 000 000 000
Returning Land-water
Returning Sea-water
New Sea water
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 58.

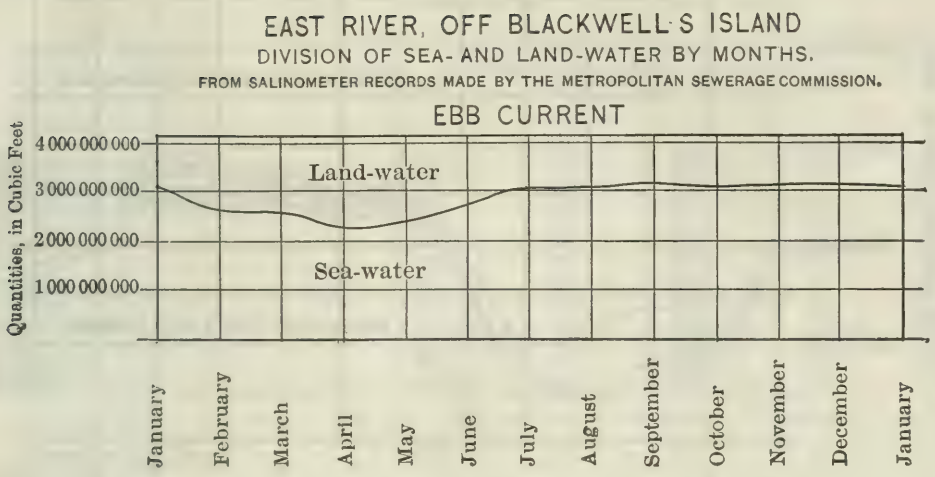

EAST RIVER, OFF BLACKWELL'S ISLAND
DIVISION OF SEA- AND LAND-WATER BY MONTHS.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.
EBB CURRENT
Quantities, in Cubic Feet
4 000 000 000
3 000 000 000
2 000 000 000
1 000 000 000
Land-water
Sea-water
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 59.

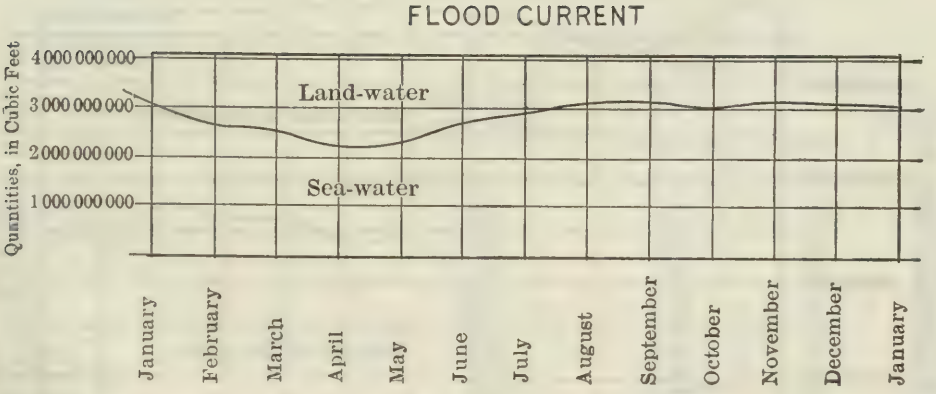

FLOOD CURRENT
Quantities, in Cubic Feet
4 000 000 000
3 000 000 000
2 000 000 000
1 000 000 000
Land-water
Sea-water
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 60.

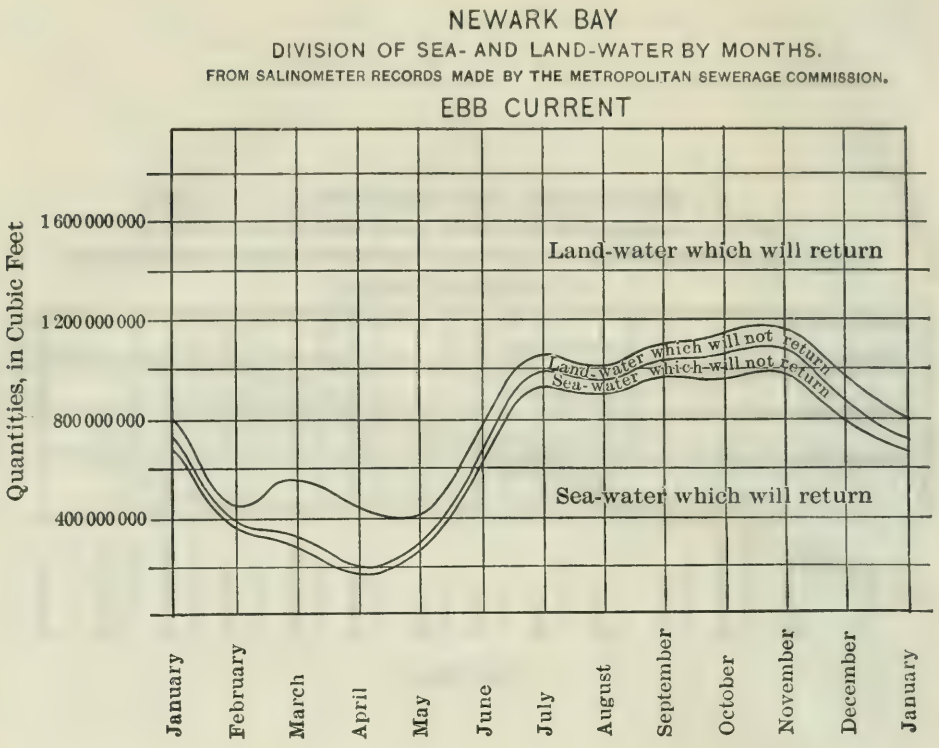

NEWARK BAY
DIVISION OF SEA- AND LAND-WATER BY MONTHS.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.
EBB CURRENT
Quantities, in Cubic Feet
1 600 000 000
1 200 000 000
800 000 000
400 000 000
Land-water which will return
Land-water which will not return
Sea-water which will not return
Sea-water which will return
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 61.

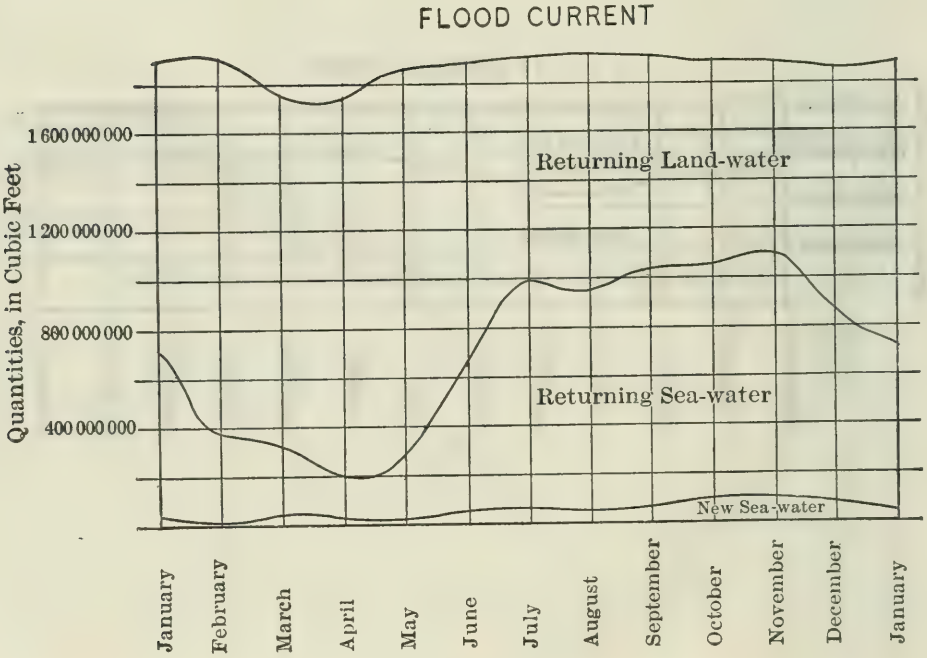

FLOOD CURRENT
Quantities, in Cubic Feet
1 600 000 000
1 200 000 000
800 000 000
400 000 000
Returning Land-water
Returning Sea-water
New Sea-water
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 62.

TABLE 28.—Newark Bay.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
January	1 179	79	669	45	1 972
February	1 535	62	360	15	1 972
March	1 424	223	281	44	1 972
April	1 549	231	167	25	1 972
May	1 570	120	262	20	1 972
June	1 210	97	616	49	1 972
July	924	62	924	62	1 972
August	973	56	892	51	1 972
September	870	63	969	70	1 972
October	823	88	959	102	1 972
November	799	81	992	100	1 972
December	999	105	785	83	1 972
Average for Year†	1 152	106	654	60	1 972

*Shown graphically on Fig. 61.
†Average of daily readings.

TABLE 29.—Newark Bay.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
January	1 179	669	45	1 893
February	1 535	360	15	1 910
March	1 424	281	44	1 749
April	1 549	167	25	1 741
May	1 570	262	20	1 852
June	1 210	616	49	1 875
July	924	924	62	1 910
August	973	892	51	1 916
September	870	969	70	1 909
October	823	959	102	1 884
November	799	992	100	1 891
December	999	785	83	1 867
Average for Year.†	1 165	644	57	1 866

*Shown graphically on Fig. 62.
†Average of daily readings.

Interpretation of Diagrams.

The sudden rise in the curve at the Battery, in Figs. 39, 40, 41, 42, 43, and 44, is caused by the discharge from and into the East River at the Battery, and also by the discharge from and into the Kill van Kull at Staten Island near The Narrows.

The salinometer readings at the station on Governors Island were used for the Battery, as it was the nearest observation station to this place. The salinometer readings at this station were affected more by the discharge from the East River than by that from the Hudson. In consequence, it is possible that more land-water should be shown at the Battery than in Figs. 39, 40, 41, 42, 43, and 44.

The diagrams showing the conditions existing in the East River, namely, Figs. 45, 46, 47, 48, 49, and 50, are not divided up like the others to show the "water which will return" and the "water which will not return." It was not possible to divide them in this manner, first, because the East River is a strait, open at both ends to bodies of salt-water, and second, because the water-sheds draining into it were so small that if a division had been attempted, as was done with the Hudson River, the "land-water which will not return" would have been so small that it would appear to be scarcely thicker than a line on the diagrams.

Conclusions.

A study of these diagrams shows that the flushing action of the waters of the harbor is not as great as the large volumes flowing on the tides would seem to warrant. All the diagrams show that the quantity of "sea- and land-water which will not return" is not a large proportion of the quantity flowing on an ebb tide. There is always a large quantity of "water which will return" on the succeeding flood tide.

The quantity of "new sea-water" is never a large proportion of the total quantity flowing on a flood tide. This "new sea-water" is only new off places situated near the ocean, and is not new at such places as the Upper Bay, Fort Washington Point, Tarrytown, East River, or Newark Bay. At these places the "new sea-water" is really water which has not been at the place on the previous tide, but comes from some other part of the harbor. For instance, at Fort Washington Point, the "new sea-water" would be from the lower reaches of the

Hudson, or Upper Bay, or perhaps some from the East River entering the Hudson at the Battery.

The water which has a seaward trend and will not return is the quantity of land-water from the water-sheds. For places near the ocean, this seaward trend is increased by some "sea-water which will not return"; but, for places well within the harbor, such "sea-water which will not return" is replaced by sea-water (as explained previously) which comes from some other part of the harbor, and is not clean ocean-water.

The diagrams indicating the division of sea- and land-water by months show that the "sea-water which will return" is always greatest during the summer and least during the spring.

The "sea- and land-water which will not return" is least during February and the summer, and greatest during the spring.

The flushing out of the harbor, or the seaward trend, is greatest during the spring and least in February, under normal conditions.

ACKNOWLEDGMENTS.

The writer acknowledges his thanks to David C. Johnson, Jun. Am. Soc. C. E., his office assistant, for aid in making the diagrams and calculations, and for numerous valued suggestions; also to O. H. Tittmann, M. Am. Soc. C. E., and Mr. F. W. Perkins, both of the U. S. Coast and Geodetic Survey, for their courtesies in permitting an inspection of the Survey records, and for their advice and criticisms.

APPENDIX A.

Harbor Divisions.

The boundaries for the several divisions of the harbor were selected as follows: At the entrance to the Lower Bay, by a line from Hook Beacon, on Sandy Hook, tangent to the westerly end of Rockaway Beach, to Barren Island. At the mouth of the Raritan River, by a line from Ferry Point to the extreme easterly point of South Amboy. At the entrance to the Shrewsbury River, by a line from the Post Light at Spermaceti Cove southwesterly to Highlands. At The Narrows, by a line drawn from the point at the south of Fort Hamilton Reservation to the dock about ½ mile south of Fort Wadsworth Light. At the south end of the East River, by a line drawn from the Battery to the foot of State Street, Brooklyn. At the south end of the Hudson River, from Castle Garden to the south end of the docks at Communipaw. At the east end of Kill van Kull, from Constable Point to the northerly point at New Brighton, Staten Island. At the west end of Kill van Kull, from the extreme end of Bergen Point southerly to Port Richmond. At the northerly end of Arthur Kill, by a line drawn from the railroad dock just east of Port Avenue, Elizabethport, southerly to Staten Island. At the south end of Arthur Kill, by a line drawn from Ferry Point to the extreme southwest end of Staten Island. At the upper end of Newark Bay, by lines drawn from the extreme southerly end of the Meadows, between the mouths of the Passaic and Hackensack Rivers, westerly across the Passaic and southeasterly across the Hackensack to the point of the Meadows. At the east end of the East River, by a line from the light on Throgs Neck to the northerly end of Willets Point. At the southerly end of the Harlem River, by a line from the foot of East 126th Street to the northerly corner of the Bronx Kills. At the northerly end of the Harlem River, by the New York Central and Hudson River Railroad bridge. The Bronx Kills, Little Hell Gate, the west branch between Manhattan and Randalls and Wards Islands, Newtown Creek, and Wallabout Bay are included in the East River. The areas of all islands have been subtracted.

APPENDIX B.

Calculation of Volumes Flowing on Tides.

Figures in Cubic Feet per Tidal Flow.

Hudson River, off the Battery:

The ebb flow equals the ebb off 39th Street plus the tidal prism between the Battery and 39th Street, Manhattan.

Ebb: 6 990 000 000 + 3.7 sq. miles × (5 280)² × 4.3 ft., mean rise of tide	7 430 000 000
Mean discharge of fresh water in 12 lunar hours	1 087 000 000
Flood: Taken as the difference	6 343 000 000

Hudson River, off Fort Washington Point:

The ebb flow equals the ebb off 39th Street, minus the tidal prism between Fort Washington Point and 39th Street, Manhattan.

Ebb: 6 990 000 000 — 6.6 sq. miles × (5 280)² × 4.15 ft., mean rise of tide	6 230 000 000
Mean discharge of fresh water in 12 lunar hours	1 087 000 000
Flood: Taken as the difference	5 143 000 000

Hudson River, off Tarrytown:

The ebb flow equals the ebb off Fort Washington Point, minus the tidal prism between Tarrytown and Fort Washington Point.

Ebb: 6 230 000 000 — 21.0 sq. miles × (5 280)² × 3.85 ft., mean rise of tide	3 980 000 000
Mean discharge of fresh water in 12 lunar hours	1 087 000 000
Flood: Taken as the difference	2 893 000 000

Newark Bay:

The figures were taken from those of the Coast and Geodetic Survey, August 14th, 1908, as they contained an estimate of the flow through Arthur Kill.

The average of flood and ebb:	
Through Kill van Kull	1 600 000 000
Through Arthur Kill	319 000 000
(1) Total, average for Newark Bay	1 919 000 000
(2) Mean fresh water discharged into Newark Bay during 6 lunar hours	52 757 000
Mean ebb flow, (1) + (2)	1 972 000 000
Mean flood flow, (1) — (2)	1 866 000 000

APPENDIX C.

Bernoulli's Theorem.

Bernoulli's theorem, applied to a strait connecting two large bodies of water, becomes, where friction is taken into account,

$$\frac{V_1^2}{2} = g\,(\zeta_{,} - \zeta_{,,}) - \zeta' \frac{P_1}{\Omega_1} \frac{V_1^2}{2} L_1 - \zeta' \frac{P_2}{\Omega_2} \frac{V_2^2}{2} L_2 - \ldots\ldots$$

Where V_1 = the velocity at the smallest section;
V_2, etc. = the velocities at other sections;
$\zeta_{,}$ and $\zeta_{,,}$ = the heights of the water each side of the smallest section;
ζ' = a friction factor;
P = the wetted perimeter;
Ω_1 = the area of the smallest cross-section;
Ω_2, etc. = areas of other cross-sections;
L_1 = the length of the channel for the smallest section;
L_2, etc. = lengths of other parts of the channel.

The conditions of continuity require, with the same volume flowing, the velocity to vary inversely as the cross-sections,

$$\frac{V_2}{V_1} = \frac{\Omega_1}{\Omega_2}; \frac{V_3}{V_1} = \frac{\Omega_1}{\Omega_3}; \text{ etc.}$$

$$V_2 = V_1 \left(\frac{\Omega_1}{\Omega_2}\right); V_3 = V_1 \left(\frac{\Omega_1}{\Omega_3}\right).$$

Substituting in the first equation,

$$\frac{V_1^2}{2} = g\,(\zeta_{,} - \zeta_{,,}) - \zeta' \frac{P_1}{\Omega_1} \frac{V_1^2}{2} L_1 - \zeta' \frac{P_2}{\Omega_2} \frac{V_1^2}{2} L_2 \left(\frac{\Omega_1}{\Omega_2}\right)^2 - \ldots.$$

Solving for V_1,

$$V_1^2 = \frac{2\,g\,(\zeta_{,} - \zeta_{,,})}{1 + \zeta' \frac{P_1}{\Omega_1} L_1 + \zeta' \frac{P_2}{\Omega_2} L_2 \left(\frac{\Omega_1}{\Omega_2}\right)^2 + \ldots\ldots\ldots}$$

Note.—The value of ζ' is about 0.007, according to report of U. S. Coast and Geodetic Survey, 1907, Appendix 6, page 425.

APPENDIX D.

TABLE 30.—Specific Gravity and Temperatures. New York Harbor Waters.

Specific Gravity at 60° Fahr. Temperatures in Degrees, Fahrenheit.

	Ambrose Channel Light-Vessel.	West Bank Station.	Fort Wadsworth Station.	Robbins Reef Station.	Governors Island Station.	Blackwells Island Station.	Throgs Neck Station.	Fort Washington Point Station.	Tarrytown Station.	Passaic Light Station.	Great Beds Station.
January, 1909.											
Temperature, Max.	45	44	43	43	41	44	40	41	37	38	41
" Min	38	34	30	35	32	32	30	27	33	30	30
Specific Gravity, Max.	1.0256	1.0219	1.0196	1.0299	1.0299	1.0204	1.0219	1.0159	1.0069	1.0140	1.0204
" " Min.	1.0239	1.0169	1.0145	1.0149	1.0199	1.0149	1.0209	1.0039	1.0014	1.0029	1.0094
" " Avg.	1.02467	1.02021	1.01778	1.01873	1.01728	1.01838	1.02100	1.00951	1.00389	1.00854	1.01706
February, 1909.											
Temperature, Max	43	43	47	44	42	42	41	42	32	44	44
" Min	38	31	28	35	31	34	29	32	42	30	29
Specific Gravity, Max.	1.0254	1.0214	1.0190	1.0209	1.0189	1.0199	1.0216	1.0129	1.0129	1.0134	1.0190
" " Min.	1.0149	1.0089	1.0043	1.0059	1.0059	1.0059	1.0179	1.0000	1.0000	1.0009	1.0035
" " Avg.	1.02312	1.00613	1.01196	1.01390	1.01478	1.01482	1.02005	1.00390	1.00650	1.00432	1.01296
March, 1909.											
Temperature, Max.	44	42	43	45	43	49	43	45	42	50	45
" Min.	38	32	30	39	30	35	35	32	33	30	33
Specific Gravity, Max.	1.0250	1.0219	1.0193	1.0201	1.0159	1.0190	1.0209	1.0140	1.0000	1.0124	1.0175
" " Min.	1.0164	1.0099	1.0089	1.0090	1.0070	1.0099	1.0179	1.0000	1.0034	1.0000	1.0084
" " Avg.	1.02244	1.01625	1.01241	1.01428	1.01269	1.01555	1.01979	1.00406	1.00034	1.00418	1.01335
April, 1909.											
Temperature, Max.	50	50	54	50	53	53	55	54	53	62	56
" Min.	42	38	39	45	42	42	40	40	40	40	43
Specific Gravity, Max.	1.0252	1.0191	1.0149	1.0161	1.0161	1.0181	1.0201	1.0052	1.0000	1.0063	1.0156
" " Min.	1.0152	1.0088	1.0043	1.0075	1.0041	1.0082	1.0181	1.0000	1.0000	1.0000	1.0021
" " Avg.	1.02229	1.01373	1.00878	1.01098	1.00899	1.01366	1.01904	1.00132	1.00000	1.00242	1.01044

TABLE 30. (*Continued.*)

	Ambrose Channel Light-Vessel.	West Bank Station.	Fort Wadsworth Station.	Robbins Reef Station.	Governors Island Station.	Blackwells Island. Station.	Throgs Neck Station.	Fort Washington Point Station.	Tarrytown Station.	Passaic Light Station.	Great Beds Station.
May, 1909.											
Temperature, Max.	62	59	65	62	63	60	63	70	62	65	65
" Min.	47	46	46	50	47	48	48	47	48	40	47
Specific Gravity, Max.	1.0243	1.0186	1.0154	1.0157	1.0147	1.0177	1.0197	1.0052	1.0000	1.0101	1.0173
" " Min.	1.0152	1.0094	1.0049	1.0080	1.0046	1.0102	1.0182	1.0005	1.0012	1.0000	1.0017
" " Avg.	1.02112	1.01203	1.01014	1.01144	1.01018	1.01440	1.01896	1.00230	1.00003	1.00339	1.01153
June, 1909.											
Temperature, Max.	70	70	74	71	71	71	71	80	71	82	72
" Min.	58	59	60	62	59	60	59	61	63	60	63
Specific Gravity, Max.	1.0240	1.0200	1.0169	1.0197	1.0177	1.0181	1.0248	1.0113	1.0038	1.0129	1.0176
" " Min.	1.0180	1.0155	1.0112	1.0123	1.0094	1.0145	1.0189	1.0031	1.0006	1.0032	1.0103
" " Avg.	1.02174	1.01800	1.01540	1.01650	1.01512	1.01656	1.01967	1.00636	1.00172	1.00822	1.01559
July, 1909.											
Temperature, Max.	72	72	74	72	75	71	73	79	75	82	78
" Min.	60	63	67	67	67	65	65	69	69	67	68
Specific Gravity, Max.	1.0245	1.0235	1.0216	1.0222	1.0205	1.0192	1.0215	1.0156	1.0068	1.0172	1.0194
" " Min.	1.0218	1.0180	1.0160	1.0156	1.0147	1.0157	1.0196	1.0057	1.0022	1.0075	1.0156
" " Avg.	1.02328	1.02069	1.01836	1.01960	1.01836	1.01805	1.02046	1.01098	1.00495	1.01226	1.01776
August, 1909.											
Temperature, Max.	73	72	75	74	74	73	74	77	75	78	76
" Min.	64	65	68	69	69	68	67	68	69	65	66
Specific Gravity, Max.	1.0244	1.0226	1.0216	1.0224	1.0205	1.0195	1.0216	1.0155	1.0090	1.0191	1.0201
" " Min.	1.0211	1.0182	1.0164	1.0167	1.0160	1.0179	1.0136	1.0083	1.0049	1.0044	1.0091
" " Avg.	1.02321	1.02041	1.01729	1.01963	1.01879	1.01862	1.02063	1.01245	1.00641	1.01189	1.01759

TABLE 30. (*Concluded.*)

	Ambrose Channel Light-Vessel.	West Bank Station.	Fort Wadsworth Station.	Robbins Reef Station.	Governors Island Station.	Blackwells Island Station.	Throgs Neck Station.	Fort Washington Point Station.	Tarrytown Station.	Passaic Light Station.	Great Beds Station.
September, 1909.											
Temperature, Max	71	70	72	72	73	71	72	73	71	74	74
" Min	64	64	61	64	62	67	63	64	63	61	64
Specific Gravity, Max	1.0240	1.0230	1.0214	1.0249	1.0212	1.0210	1.0217	1.0174	1.0084	1.0205	1.0206
" " Min	1.0221	1.0196	1.0160	1.0189	1.0171	1.0170	1.0194	1.0092	1.0049	1.0091	1.0166
" " Avg	1.02314	1.02095	1.01791	1.02038	1.01945	1.01897	1.02073	1.01281	1.00674	1.01311	1.01844
October, 1909.											
Temperature, Max	64	64	65	67	65	70	67	66	65	65	64
" Min	51	49	48	52	51	51	49	48	46	44	46
Specific Gravity, Max	1.0244	1.0232	1.0205	1.0250	1.0210	1.0205	1.0222	1.0154	1.0075	1.0193	1.0203
" " Min	1.0224	1.0195	1.0159	1.0180	1.0157	1.0167	1.0195	1.0095	1.0051	1.0088	1.0176
" " Avg	1.02348	1.02106	1.01916	1.02009	1.01882	1.01859	1.02111	1.01238	1.00656	1.01340	1.01903
November, 1909.											
Temperature, Max	55	52	53	54	55	56	56	55	51	55	51
" Min	48	43	42	46	43	43	39	40	40	40	39
Specific Gravity, Max	1.0249	1.0233	1.0198	1.0220	1.0206	1.0201	1.0230	1.0163	1.0088	1.0191	1.0196
" " Min	1.0229	1.0202	1.0152	1.0184	1.0151	1.0173	1.0196	1.0092	1.0052	1.0091	1.0179
" " Avg	1.02384	1.02120	1.01933	1.02007	1.01872	1.01896	1.02168	1.01266	1.00665	1.01367	1.01878
December, 1909.											
Temperature, Max	49	43	43	45	45	46	46	42	41	44	40
" Min	37	31	29	34	30	31	32	30	32	31	27
Specific Gravity, Max	1.0246	1.0219	1.0194	1.0226	1.0205	1.0210	1.0230	1.0169	1.0079	1.0159	1.0224
" " Min	1.0234	1.0189	1.0146	1.0159	1.0144	1.0159	1.0209	1.0081	1.0039	1.0024	1.0179
" " Avg	1.02384	1.02361	1.01866	1.01940	1.01750	1.01992	1.02210	1.01111	1.00589	1.01053	1.01911

DISCUSSION

Mr. Hazen.

ALLEN HAZEN, M. AM. SOC. C. E.—It is extremely fortunate that the observations recorded in this paper have been made, and that they have been compiled and put in convenient and readily understood shape. The author's method of analysis follows closely that used by the speaker in discussing the same problem* seven years ago. At that time the data for salinity, tidal flow, and fresh-water flow, were meager, and there was no information as to the extent of underrun. The figures then obtained, with limited data, in comparison with those of the author, are given in Table 31. The author's figures afford a striking confirmation of the substantial accuracy of those used by the speaker in discussing the sewage disposal problem in the harbor at that time, and in a professional report made soon afterward.

TABLE 31.

	Annual Average Figures:	
	Hazen, 1906.	Parsons, 1913.
Ebb Tide:		
Percentage of river-water on its last trip out	7	9.8
Percentage of river-water which will come back	28	25.0
Percentage of sea-water on its last trip out	14	18.4
Percentage of sea-water which will come back	51	46.8
Flood Tide:		
Percentage of new sea-water which has never been in the harbor before	23	20.5

The figures in Tables 18 and 19, containing corresponding data by months, are most interesting. The quantity of new water passing The Narrows at each tide by months, from these tables, and the corresponding flow of a stream necessary to furnish an equal quantity of new water in the course of the 24 hours, are as shown in Table 32.

The figures in Table 32 show that 65% of the new water in the harbor throughout the year is sea-water, and that 35% is land-water. For every cubic foot of water that goes through The Narrows from all the rivers draining 14 395 sq. miles, 2 cu. ft. of water come in from the ocean, circulate up and down the harbor, and go out to sea again. The exchange of water in the harbor is as great as that which would result from the fresh water from an area of 41 000 sq. miles. Practically, it is equivalent to much more, because, in a river, the flow is variable, and there is a period in summer when it is much less than the average. In New York Harbor there is, practically speaking, no dry-weather period. When the flow of fresh water is less, the flow of sea-water is greater, and the total flow is maintained so that the

* *Journal*, Assoc. of Eng. Societies, Vol. XXXVI, p. 293.

Mr. Hazen.

minimum is a larger percentage of the average than would be the case in any natural river.

The author's figures indicate a minimum flow for February of 30 000 cu. ft. per sec., between much larger values for January and March. The low figure for February is due to a very low figure for sea-water exchanged during that month. The reasons for fluctuations in the volume of land-water are obvious, but there is no apparent reason why the quantity of sea-water should fluctuate to as great an extent as is indicated by Table 32. The tide must have risen and fallen during that month as in other months, and the opportunities for exchange of sea-water were the same as at other times. When the rivers are in flood, the quantity of fresh water entering the harbor is greatly increased, and tends to take the place of, and keep out, an equal volume of salt water. For the February in question no such condition existed. On the other hand, the land-water was nearly down to the low figure for the year. The speaker thinks that the explanation of this apparently low February flow is to be found in some matters, not sufficiently taken into account in the calculation, which, perhaps, cannot be taken into account at the present time.

TABLE 32.—NEW WATER IN THE HARBOR WITH EACH TIDE, IN MILLIONS OF CUBIC FEET, DURING EBB TIDE.

Month.	Land-water.	Sea-water.	Total.	Totals, in cubic feet per second.
January	880	2 240	3 120	70 000
February	690	670	1 360	30 000
March	2 500	2 510	5 010	112 000
April	2 590	1 740	4 330	97 000
May	1 340	1 050	2 390	54 000
June	1 090	1 780	2 870	64 000
July	700	2 180	2 880	65 000
August	620	1 520	2 140	48 000
September	710	2 150	2 860	64 000
October	990	3 610	4 600	103 000
November	910	3 190	4 100	92 000
December	1 070	3 470	4 540	102 000
Averages	1 180	2 210	3 390	76 000

The assumption is made, if the speaker understands correctly, that the quantity of water passing through The Narrows during each month is the same as that of the fresh water flowing in all the tributary rivers. This assumption must be more or less in error, because, obviously, there is retardation of the flow of fresh water through the large and variable reservoir of brackish water between The Narrows and the farthest points to which salt water extends. This retardation must have the effect of modifying, probably to a substantial extent, the quantity of fresh water flowing through The Narrows during any

Mr. Hazen.

calendar month. If these differences could be adequately taken into account and allowed for, the speaker suggests that the monthly variation of the combined flow might be found to be more nearly constant than is indicated by the author's tables. Taking this matter into account probably would not greatly change the annual average figure. It would have a steadying effect on the monthly flows, and reduce the fluctuations.

If it is assumed that the lowest months in the author's tables may be accounted for in this way, it appears that the exchange of water in the harbor is equivalent to a steady flow of between 50 000 and 100 000 cu. ft. per sec., averaging 76 000 cu. ft. per sec. In comparison with this, the speaker, in 1906, on the basis of the few data then available, estimated the corresponding flow at 68 000 cu. ft. per sec. No river in the United States has a minimum flow as great as this. Probably no other great city in the world is as favorably situated as New York on rivers having currents of such volume and strength.

It has been sometimes assumed that a flow of 3 cu. ft. per sec. would suffice to dilute the sewage from 1 000 people enough to make it inoffensive. This ratio is not everywhere accepted, but, taking it as a basis, if the flow through The Narrows could be fully utilized for dilution, it would serve to carry off the sewage of from 17 000 000 to 34 000 000 people. Even if a larger relative dilution is assumed to be necessary, as, for instance, 4, or even 5 cu. ft. per sec., per 1 000 of population, it is still adequate to serve a population which will not be attained for many years.

One of the practical difficulties, in the utilization of this great water quantity for dilution, is to arrange matters so that the sewage will be mixed evenly with it. At present more than a proportionate quantity of sewage is discharged into some branches of the harbor, as, for instance, into the East River and the Harlem River, and these branches have a higher relative pollution and are nearer to a permissible limit than is the harbor as a whole.

The figures for tidal flow, put in convenient and accessible form in this paper, will be of the greatest service in future discussions of this most important question.

Mr. Thomson.

T. KENNARD THOMSON, M. AM. SOC. C. E.—The Society owes a hearty vote of thanks to Mr. Parsons for this very valuable and interesting paper. It is especially interesting to the speaker on account of his plan for relieving the congestion in New York Harbor.

This plan has already been brought to the attention of the Society,* and this opportunity is taken to show the same plan with an addition, Fig. 63, which has been seriously suggested by a very able man.

* "The Problem of the Lower West Side Manhattan Water-Front of the Port of New York," by B. F. Cresson, Jr., M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 268.

Mr. Thomson.

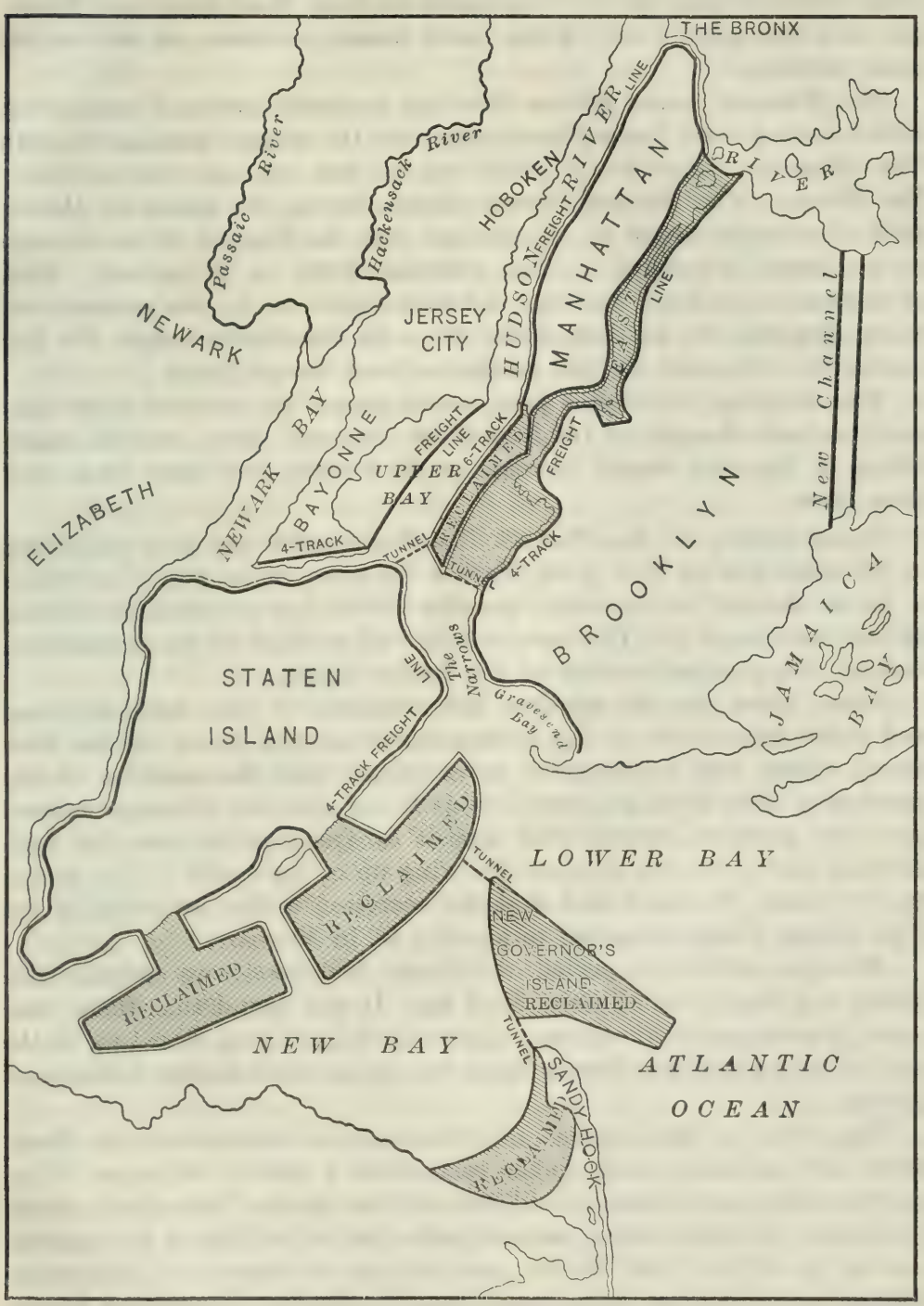


FIG. 63.

Mr. Thomson.

The plan consists of the reclamation of the East River, and the building of a serviceable channel between Flushing and Jamaica Bays. This would reclaim about 10 sq. miles between Manhattan and Brooklyn, and also solve many of the rapid transit problems, as well as the dock problems.

Mr. Parsons' paper proves that the speaker's original proposition would have a very insignificant effect on the waters passing through The Narrows, stated to be 75 000 cu. ft. per sec., as referred to by Mr. Hazen. The Mohawk River, alone, during the floods of March, 1913, discharged 96 000 cu. ft. per sec. into the Hudson River, though its discharge in summer is often less than 2 000 cu. ft. per sec. This, of course, is unusual, but the ordinary variation in the quantity of water entering the Hudson River from its tributaries makes the displacement, suggested by the speaker's plans, insignificant.

The carrying out of this plan would insure the greatest boom that has ever been thought of for New York City and State, and the population of the city would increase to 12 000 000 and more in a very short time.

Mr. Owen.

JAMES OWEN, M. AM. SOC. C. E.—The speaker has been interested in the tidal flow in New York Harbor for some years, and there seems to be an element in the whole question which has not been considered by any one except Mr. Thomson, who is rather ahead of the proposition, that is, the gradual closing of the harbor itself.

Some years ago the speaker had occasion to take some evidence and make some tests on the encroachment on the shores on the New Jersey Coast, and incidentally he examined into the question of the opening of New York Harbor. It was found that two elements entered into that question, namely the deposit by fresh water from the high grounds above, and the gradual throwing up of the sands by the storms on the coast. It was found that the accretion at the estuaries of the New Jersey Coast amounted to about 3 ft. in 30 years.

At some prehistoric period, the Jersey Meadows were covered with water, and they have all been filled up. It was found that there were some 21 estuaries on the Jersey Coast which have been closed up in the last 150 years, and that Sandy Hook was at one time merely a cluster of islands.

The report of the Jamaica Bay Commission showed that the Rockaway Spit had encroached to the east about 1 mile in 30 years. Considering this encroachment and that of the Sandy Hook Spit, which is about in the same ratio, one may note the probability of the gradual closing up of New York Harbor, not only by the deposits of the waters themselves—which are very small from the Hudson River—but by the encroachment of the sea.

Whatever experiments are made will have to be carried along year by year, counting on the encroachments of the ocean. It is a very

important question, this gradual closing up of New York Harbor by the encroachment of the sea. Mr. Owen.

KENNETH ALLEN, M. AM. SOC. C. E. (by letter).—The author has described the tidal condition existing in New York Harbor so fully that little remains to be added. The special application of tidal studies to those of sewage disposal is obvious. Dilution of a sewage effluent may be secured by the variable currents induced by winds and differences of temperature, as in cities discharging sewage into a lake; but, in general, dependence is placed on the much more reliable and effective currents due to the flow of upland waters in rivers or the tides of the sea. In the case of New York City, though large dependence is placed on the upland waters discharged by the Hudson River, the condition of the East River and Upper Bay depends in a great degree on the refreshing influence of the new sea water brought in twice a day by the tides. To a much greater extent is this true of the two shallow bays—Newark and Jamaica—the waters of which receive large volumes of sewage, but yet contain a fair amount of dissolved oxygen brought in by the proportionately large tidal prism at every tide. Owing to the existence of such connecting waterways as the East and Harlem Rivers, Little Hell Gate, Bronx and Arthur Kills, and Kill van Kull, combined with the varying conditions of flow in the Hudson River, the tidal phenomena of New York Harbor are as complex as could well be imagined, and a reasonable approach to true average conditions can only be found by a long series of observations. In the case under discussion, much information was available from past observations of the U. S. Coast Survey and the War Department, but, especially with reference to the East and Harlem Rivers, this was incomplete and has but recently been secured by Col. W. M. Black, Corps of Engineers, U. S. A. Mr. Allen.

Aside from velocities and discharges at certain points or cross-sections, information was desired by the Metropolitan Sewerage Commission as to the probable dispersion and drift of floating solids if set adrift at specific points, and though the float observations which were undertaken provided a certain amount of general information regarding probable velocities, on which discharges might be estimated, dependence could not be placed on these alone for the latter purpose, owing to the manifest discrepancies due to local or temporary conditions.

Float observations, however, are valuable as an indication of the probable trend of sewage (which in salt water rises to near the surface), its oscillation back and forth with the tides, and of the shores on which its floating constituents may strand.

In 1897 the writer made a series of float observations for the Baltimore Sewerage Commission* in Chesapeake Bay off North Point, to

* Report, Baltimore Sewerage Commission, 1897, Mendes Cohen, Past-President, Am. Soc. C. E., Chairman.

Mr. Allen.

determine the most favorable location for the discharge of sewage. At six selected points, about ½ mile apart and from ⅝ to 2½ miles from shore, poles were set in the bottom, which was nowhere more than about 20 ft. from the surface at low water, the tidal range being about 1 ft.

A 30-ton schooner was chartered and anchored in the vicinity of the work as headquarters for the party, which consisted of an observer, a recorder, the captain, mate, and cook. A small steam launch, with captain and engineer, was secured with which to follow the floats. For communication with the party a tug was engaged.

The floats consisted of 2 by 2-in. by 7-ft. yellow pine sticks weighted on the lower end by cast-iron washers strung on a wire until only 5 or 6 in. of the other end were left above water. A ¼-in. iron rod was inserted in this upper end on which a 12 by 12-in. red or black flag was attached bearing a serial number in white. These were set adrift at the several points in succession, beginning at about 6 A. M. and continuing at stated intervals, by a man in a rowboat. They were observed by sextant as often as practicable from the launch, passing from one float to another until dark. No attempt was made to recover the floats, which finally went down the bay or grounded on shoals.

In this way 150 floats were set adrift during an entire month, May 27th to June 26th, so as to include all phases of the moon and tide.

A tide gauge was set up at a neighboring light-house and a record of the tides and wind was kept during the work.

The cost of these observations was:

Schooner, crew, and launch	$296.98
Salaries	378.50
Provisions	53.91
Coal for launch	32.00
Floats	70.05
Supplies, fares, and miscellaneous	53.44
Plotting traces of floats on charts	54.50
Total	$939.38

On account of the unseaworthiness of the launch in bad weather 10½ days were lost.

By this method, a large number of observations were made during the day of floats set out at different points, at different times, and at different stages of the tide. The records of individual floats were not very complete, nor were they often continued beyond the day they were set adrift, but, taken together, they indicated very graphically the points likely to be reached by floating matter starting from any one of the six assumed points of outfall.

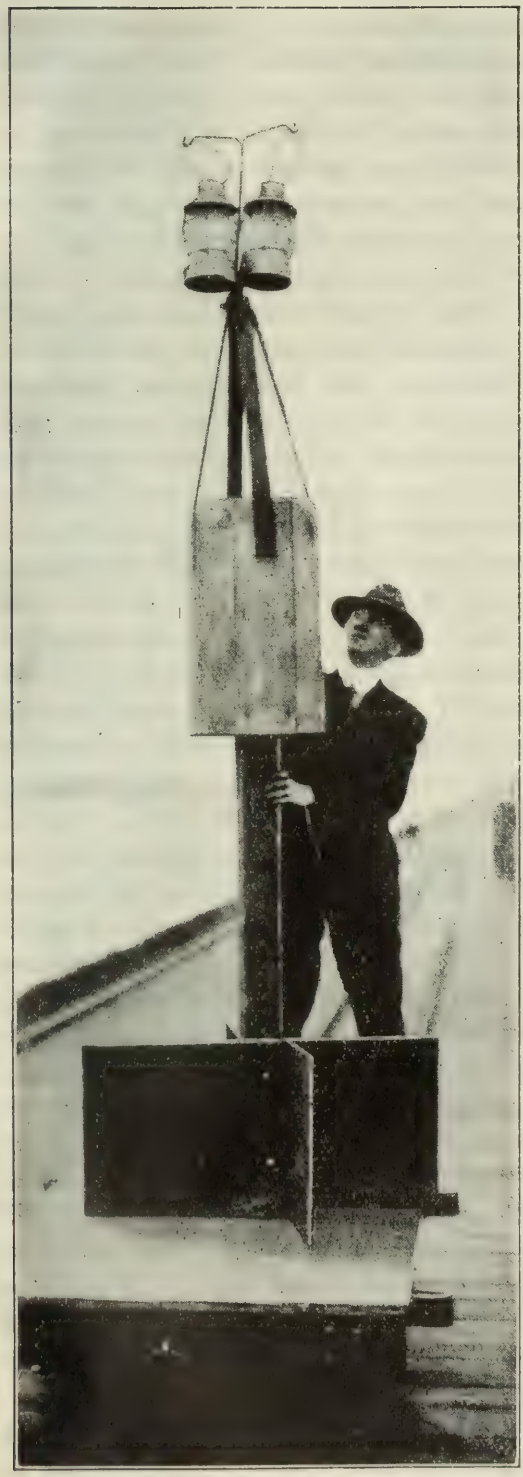


FIG. 64.—SPAR FLOAT.

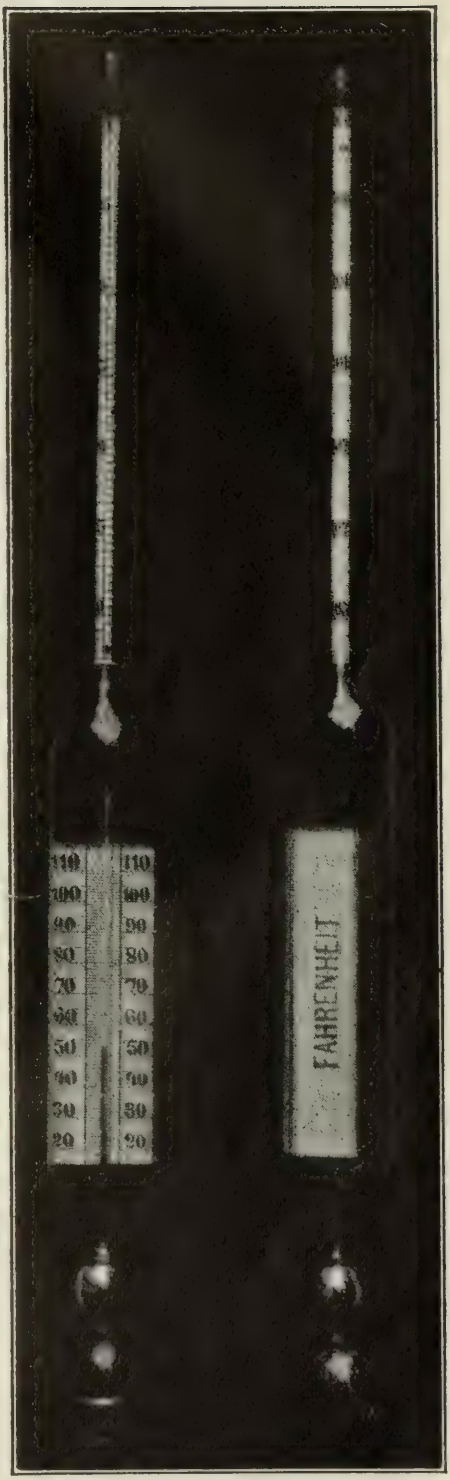


FIG. 65.—SALINOMETER.

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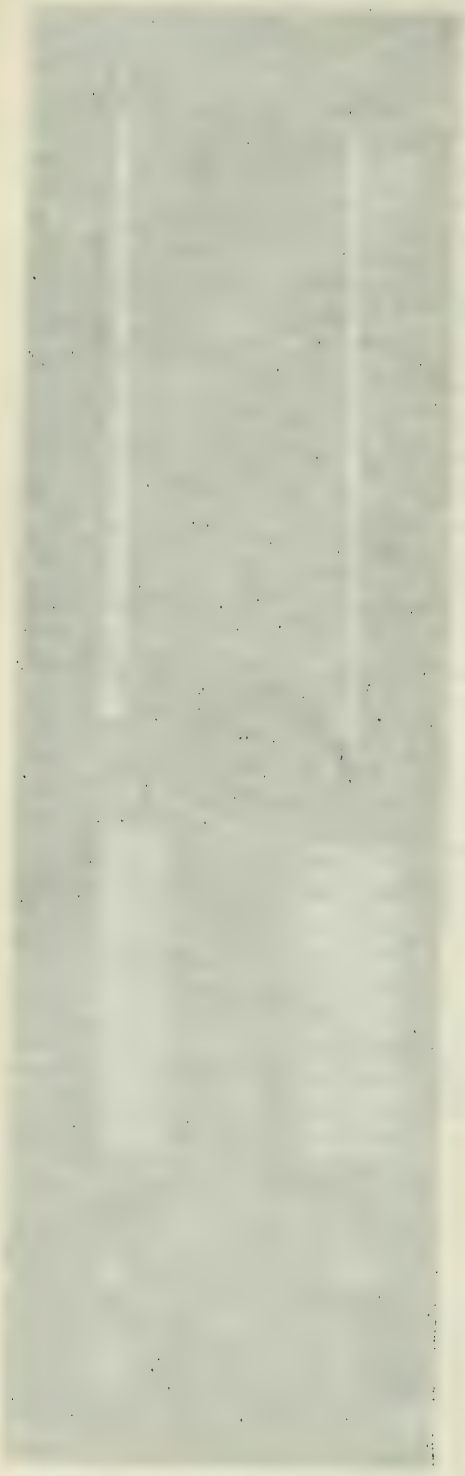


Mr. Allen.

They also indicated: (1) a general trend off shore toward the main ship channels and away from the oyster beds which were farther down the bay and near the shores, or in the estuaries; (2) that under rare conditions would floating material set out at the point finally recommended (2 miles from shore) be carried up the Patapsco River as far as Sparrows Point; and (3) that after a continuous discharge for 33 consecutive hours sewage would be dispersed in all probability over an area of some 16 sq. miles, and hence become thoroughly diluted.

The type of float selected was inexpensive and, on the whole, satisfactory, being convenient to handle and but little affected by the wind. Tests were made with black, white, and red flags, and, contrary to expectations, the black was generally discovered most easily and white the least so, although this depended on the weather. Red or black flags, appearing the same in the distance, were usually first seen with low-power field glasses at distances ranging from $\frac{1}{2}$ to 1 mile.

The New York experiments, taken up by the writer in 1908, were conducted under quite different conditions.* Considerable populations occupy the shores for many miles along the several watercourses, and the currents are so swift as to render useless any attempt to follow any number of floats with one boat. It was decided, therefore, to set out only one float at a time and observe its position at frequent intervals throughout the day. To avoid the loss of the float by the absorption of water, the double-can type, described by the author, was devised. This was found convenient, inexpensive, and in general satisfactory, except that it was easily destroyed by being run down by tugs in the crowded waterways. Also, the small flag which it was intended should be carried by the upper can made it top-heavy, and, as the observer was never far from the float, the flag was finally omitted as unnecessary.

During the following year the longer "spar" float, Fig. 64, was adopted and found well suited to the conditions to be met. It was open to the objection found with the Baltimore float of absorbing water, but, having large submerged vanes, the influence of the wind was relatively small, and there was no objection in allowing the top to project above the water sufficiently to prevent loss from this cause. The size of this float, moreover, permitted two lanterns to be carried on a $\frac{3}{8}$-in. steel rod supported by a light frame at a height of some 3 ft. or more above the water surface, and so it was possible to follow the float by night as well as by day. This was found essential in order to secure the information desired.

Locations in the open water of the bay were generally made with a sextant, as on the Chesapeake Bay work; but, in the strong currents of the rivers, and when near the shore, where locations could be identi-

* Report, Metropolitan Sewerage Commission of New York, 1910, George A. Soper, M. Am. Soc. C. E., President.

Mr. Allen. fied, an estimate of the position noted briefly in the recorder's book was found more reliable. The azimuth compass was used to a limited extent in taking bearings up or down stream to objects not too distant, such as bridge piers, but was found unreliable where the plotted position depended on the accuracy of the bearing from a distant point, and considerable care was required to see that the dial swung free.

Although these experiments were intended primarily to indicate the possible drift of floating matter, the results have been utilized in confirming or modifying existing values for the velocity at strength of tide, and for the tidal flow at different points in the harbor.

Where current velocities at specific locations were desired, these were obtained by anchoring at such locations and noting the time taken by a double-can float to drift 100 ft. The distance was measured by a fine cord attached to the float, and the time by a stop-watch.

In order to obtain continuous night-and-day observations, three motor boats were chartered, each with an observer who was to follow the float for an 8-hour shift, but the long distances traveled during a tide, the inaccessible location of much of the work, and the prevailing fogs caused so great a loss of time, and, in addition, it was found so difficult to maintain operations with small boats during stormy winter weather in the crowded thoroughfares, that a seaworthy tug was finally secured and provisioned for a week's continuous work. In this way floats were followed well out into the Lower Bay, and there was little loss of time.

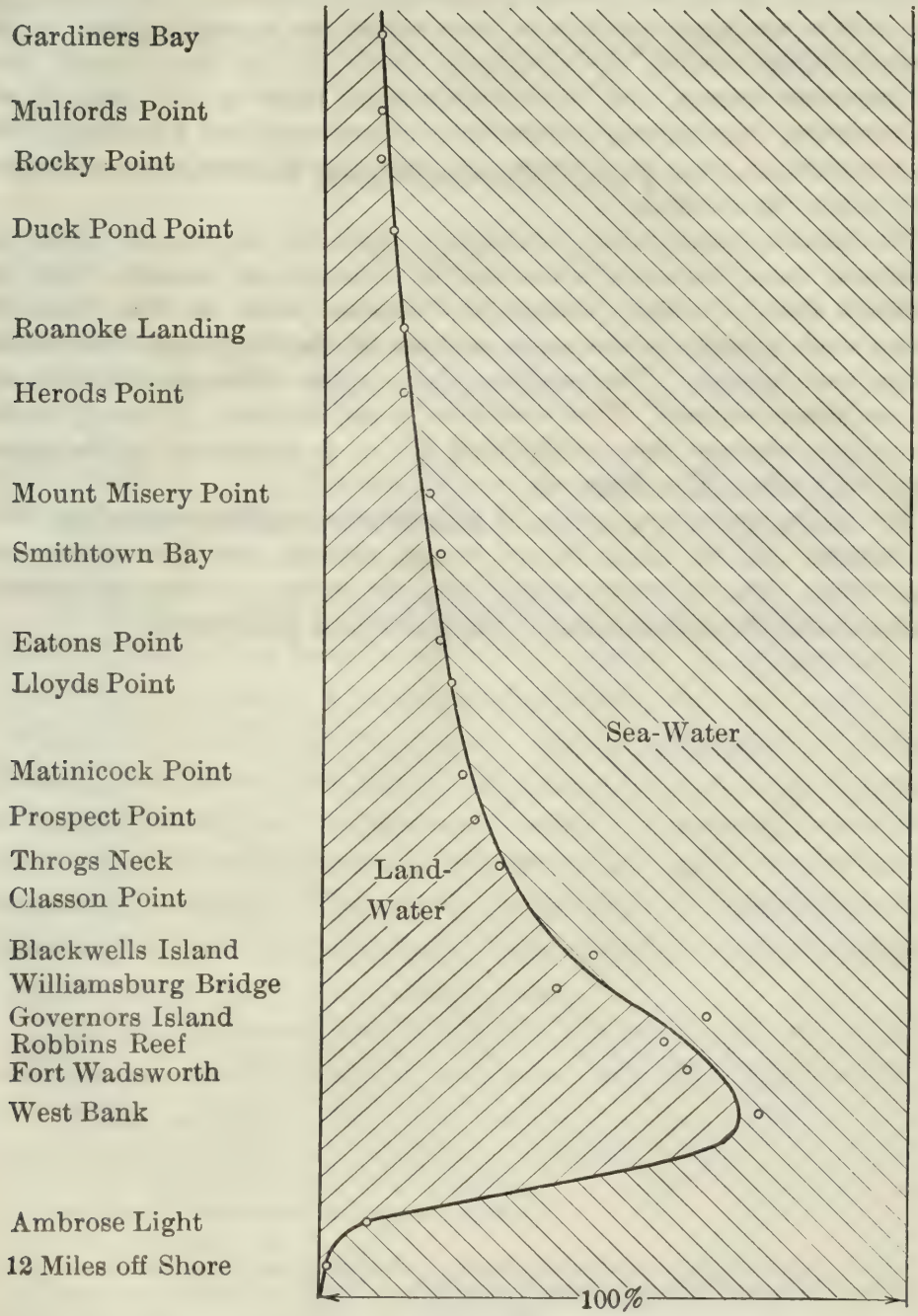
In studying the circulation of water in New York Harbor the salinometer* (Fig. 65) proved to be an important adjunct, indicating at the same time the specific gravity of the water and its temperature, by which the proportion of "normal" sea water could be determined at once by reference to a diagram. By a series extending from the eastern end of Long Island Sound through the East River to the Upper Bay, for instance, it was shown by the high salinity east of Hell Gate that the waters of the Sound were not those which had come from the Upper Bay, where the Hudson River causes the proportion of sea water to drop to 60 or 70%, but were essentially those of the Atlantic entering from the east, and hence that there exists a definite net discharge to the west and south through the East River—a fact difficult to prove before Col. Black's more recent gaugings were made.

Mr. Parsons. H. de B. Parsons, M. Am. Soc. C. E. (by letter).—The writer is pleased to note the close approximation between the percentages obtained by Mr. Hazen in 1906 and those shown in the paper, as indicated in Table 31.

Referring to Mr. Hazen's criticism of Table 32, the writer is of the opinion that the dips in the curves for February are due to the

* "The Use of the Salinometer in Studies of Sewage Disposal," by Kenneth Allen, M. Am. Soc. C. E., *Journal*, Assoc. of Eng. Societies, April, 1911.

Mr. Parsons.



PROPORTION OF SEA- AND LAND-WATER THROUGH LONG ISLAND SOUND AND NEW YORK HARBOR, APRIL 13TH AND 15TH, 1909.

FIG. 66.

Mr. Parsons.

salinometer readings being taken throughout the year 1909 and used in connection with the average monthly discharges of the rivers. If salinometer readings could have been taken for as many years as the stream-gauging record, the diagrams shown in the paper would have had smoother curves, and the corresponding tables would have shown less variation from month to month. It happened that February, 1909, was a month of great river discharge, and March, 1909, a month somewhat below the normal.

Mr. Hazen refers to an assumption made by the writer. This assumption, more correctly speaking, is a condition, namely, that the resultant flow of water passing any section, such as The Narrows, during each month, is the same as that of the land-water discharged during such month. The resultant flow is the difference between the ebb and flood volumes. This condition is self-evident. The only variation is the resultant flow of the East River, as explained in the paper, which will affect The Narrows.

Mr. Allen refers to a series of salinometer readings extending from the eastern end of Long Island Sound through the East River and Upper Bay. Fig. 66 shows graphically the results of these salinometer readings and the proportions of sea-water and land-water.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1279

KINETIC EFFECTS OF CROWDS.*

By C. J. Tilden, Assoc. M. Am. Soc. C. E.†

With Discussion by Messrs. Frazer C. Hilder, Henry H. Quimby, R. D. Coombs, J. B. French, and C. J. Tilden.

For nearly a century the accepted value for the weight of a dense crowd of people has been about 100 lb. per sq. ft., conservative designers often assuming a slightly higher figure, and their more daring brethren a considerably lower one.‡ The investigations of Stoney,§ Kernot,‖ and others, long ago showed that this was by no means the maximum value, and the elaborate work of Johnson,¶ published in 1904, showed that an intensity of 183 lb. per sq. ft. was within the range of possibility. The effect of Johnson's investigations was to raise slightly the load intensities prescribed by some specifications, but this effect was by no means general.

Thus far, the purely static effect of a crowd is the only one that has received careful study by engineers, and the "dead weight" is the loading assumed. The fact that this is not sufficient in considering the load that may come on a bridge or other structure was

* Presented at the meeting of April 16th, 1913.

† Now M. Am. Soc. C. E.

‡ "A Memoir on Suspension Bridges." Charles S. Drewry, London, 1832, p. 113.
"Treatise on Bridge Building." S. Whipple, Utica, N. Y., 1847, Essay No. II, p. 115.
"Encyclopedia of Architecture." Gwilt. p. 429.
For modern load requirements in specifications of highway bridges, etc., see *Transactions*, Am. Soc. C. E., Vol. LIV, p. 479; *Engineering News*, Vol. 65, p. 392, pp. 440 and 460; Vol. 66, pp. 61 and 643; Vol. 68, p. 824, etc., etc.; Illinois Highway Commission. Third Report, p. 121.

§ "Theory of Strains in Girders and Similar Structures." B. B. Stoney, London, Revised Edition, 1886, p. 616.

‖ See Editorial, *Engineering News*, Vol. 29 (March 16th, 1893), p. 252.

¶ *Engineering News*, Vol. 51, p. 360; *Transactions*, Am. Soc. C. E., Vol. LIV, p. 441; *Harvard Engineering Journal*, Vol. IV (April, 1905), p. 29.

recognized by Robert Stevenson in the early part of the last century. In his paper entitled "An Account of Suspension Bridges", published in 1821,* is the following interesting paragraph:

"But the effect we have to provide against in bridges of suspension, is not merely what is technically termed *dead weight.* A more powerful agent exists in the sudden impulses, or jerking motion of the load, * * *. The greatest trial, for example, which the timber bridge at Montrose, about 500 ft. in extent, has been considered to withstand, is the passing of a regiment of foot, marching in regular time. A troop of cavalry, on the contrary, does not produce corresponding effects, owing to the irregular step of the horses. The same observations apply to a crowd of persons walking promiscuously, or a drove of cattle," etc.

Stevenson, however, does not suggest any definite values for this "more powerful agent" which he recognizes and describes with such clear appreciation.

Shortly after Johnson's paper was published, a letter signed by Mr. R. Moreland appeared in London *Engineering,*† in which a suggestive experiment was reported:

"In 1900 my firm had a contract for the Manchester Racecourse for some large stands, and to gain information we put as many men as possible on our 10-ton weigh-bridge, and we found that we could get ninety men as close as possible into a space 14 ft. by 8 ft., or 112 square feet, and they weighed 115 cwt. 1 qr., which would equal 1 cwt. per square foot. We then asked them to jump, and the load went up to 1½ cwt.; they then ran four abreast across the machine, but no excess was recorded over the 1½ cwt. per square foot."

Details of this experiment are meager, but it is the only one, so far as the writer is aware, in which a determination is sought of the increased load effect due to motion in a group of human beings. In general, in structural work at least, this increase is assumed to be cared for by the "factor of safety", that pernicious and misnamed offspring of ignorance.

The study of kinetic effect is far more complex than that of static effect, for the movements of a crowd of people may take place in infinite variety, and each change of motion must be accompanied by some exertion of force other than the purely static condition. The problems suggested, therefore, are capable, at best, of only approximate solution. The following study is an attempt to throw some light on

* *Edinburgh Philosophical Journal*, No. 10, p. 255.

† Vol. LXXIX (April 28th, 1905), p. 551.

the question of determining and analyzing as far as possible the forces exerted by an individual under certain simple conditions of motion The extent to which such observations apply to crowds, and the limitations of such an application are then considered.

The subject is divided naturally into two parts: first, the vertical effect, as in the experiment described by Moreland and quoted above, manifested as an increase over the static or dead load; and, secondly, the horizontal effect which appears as a lateral force. The first is due to changes of motion in a vertical direction, the second to similar changes in a horizontal direction. The experiments described are simple and direct, and though the results may not be scientifically exact, they are at least instructive, and furnish a basis for modifying somewhat our views on the possible effect of crowds.

Vertical Effect.—1.—Rising Suddenly from a Crouching Posture.—In this experiment the subject was asked to assume a crouching position, as shown by Fig. 1, on an ordinary platform scale. His weight was determined and recorded. The counterpoise was then moved out on the scale-beam until it registered a load 40 or 50 lb. in excess of the static load, the hooked end of the scale-beam resting, of course, against the lower stop. The subject was then asked to rise smartly to a standing position, as shown by the dotted lines in Fig. 1, whereupon the scale-beam shot up momentarily, showing a sudden large increase in the downward load effect of the man, as a result of the upward acceleration given to his body. A somewhat larger load was then registered on the scale-beam and the operation repeated. A great deal of care was exercised to prevent the motion from becoming a jump, and the subject was warned not to let his feet leave the platform. The movement was smart and quick, however, occupying, per-

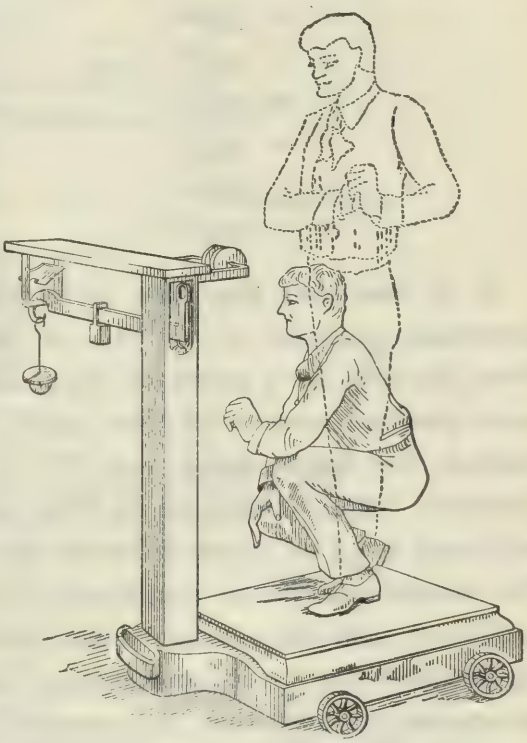


FIG. 1.

haps, $\frac{1}{3}$ or $\frac{1}{4}$ sec. Each repetition of the movement was recorded, and the words "Raised" or "Not raised", referring to the scale-beam, were written after the number representing the load registered on the scales. The following is a sample of the record sheet kept of each experiment:

(Name of subject.) Static weight = 155 lb. (Date.)

Scales set at 220 lb.	Raised.
" " " 240 "	Raised.
" " " 260 "	Barely raised?
" " " 260 "	Raised.
" " " 265 "	Raised.
" " " 270 "	Raised.
" " " 275 "	Not raised.
" " " 275 "	Raised.
" " " 280 "	?
" " " 280 "	Not raised.
" " " 280 "	Barely raised?

It is seen that this man, weighing 155 lb., exerted a momentary downward pressure of 275 lb., as shown by the scale-beam lifting from the stop, as a result of the movement described. As there was some question, even after three trials, as to whether 280 lb. had been exerted, the figure taken was 275 lb. or an increase of 77% over the static load. (See Line D, Fig. 2, I.) Eight experiments of this sort were tried on seven different men, weighing from 139 to 233 lb. The results are shown graphically by Fig. 2, I, in which the black portion of each line represents the static weight and the shaded portion the increase due to rising suddenly from a crouching position. It is interesting to note that the greatest increase recorded, 80%, occurs for the heaviest man, *G*, a former tennis champion who is very active on his feet.

The two lines marked A and A_x, one showing 58% increase and the other 67%, are for the same man. The result shown at A was obtained in the usual way, as illustrated by Fig. 1 and previously described. The case, A_x, differed from A in that a timber, about 11 in. deep and weighing 108 lb., was put on the scale platform on which the subject stood. While it is not likely that the interposition of this heavy mass had anything to do with the slight increase (from 58 to 67%) shown in A_x, it obviously did not tend to diminish the kinetic effect.

II.—Rising Suddenly from a Sitting Position.—This experiment was similar to I, and was conducted in the same manner, the only difference being that the subject rose suddenly from the chair in which he was seated, instead of getting up from a crouching position.

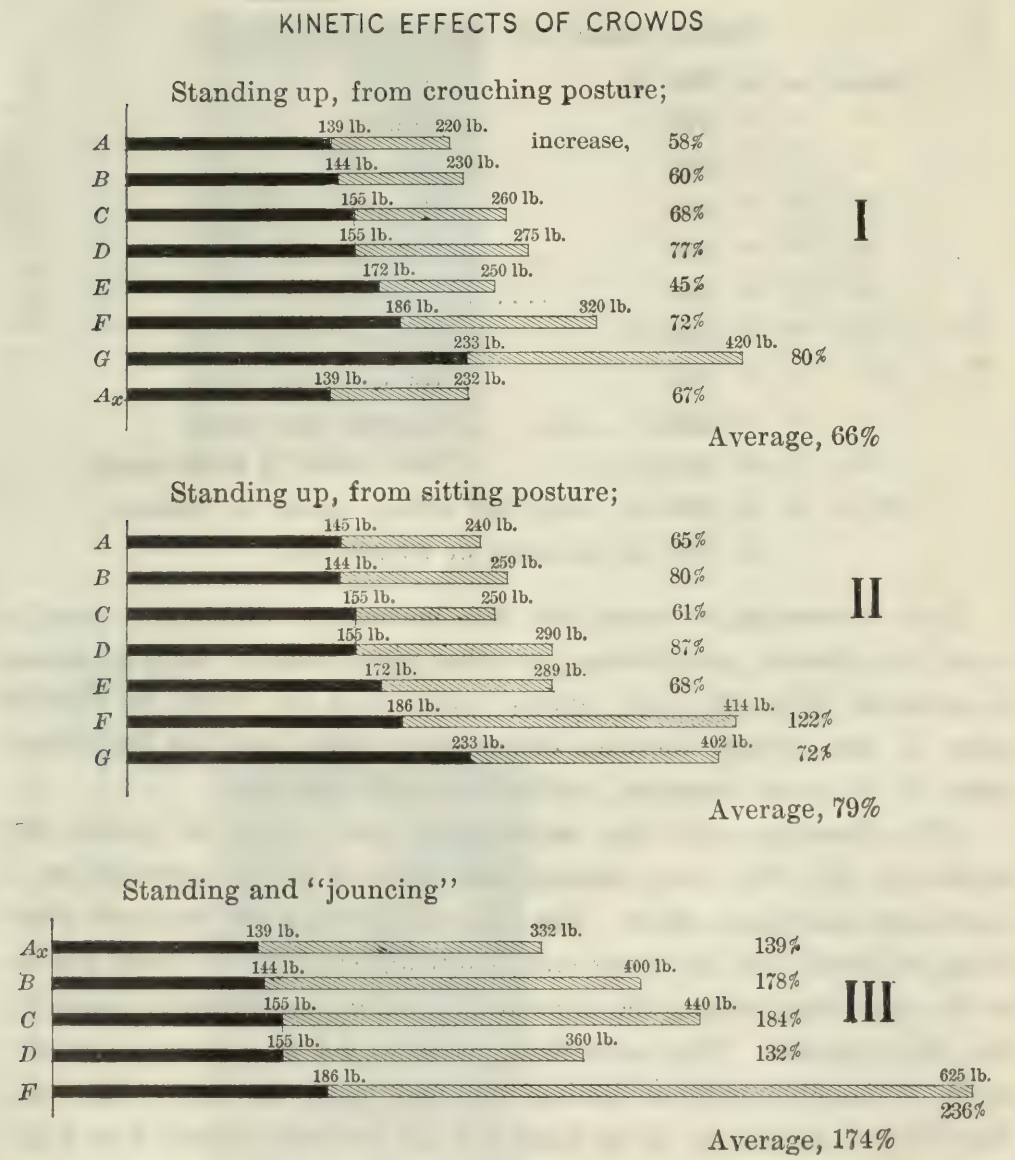

KINETIC EFFECTS OF CROWDS
Standing up, from crouching posture;
A 139 lb. 220 lb. increase, 58%
B 144 lb. 230 lb. 60%
C 155 lb. 260 lb. 68%
D 155 lb. 275 lb. 77%
E 172 lb. 250 lb. 45%
F 186 lb. 320 lb. 72%
G 233 lb. 420 lb. 80%
Ax 139 lb. 232 lb. 67%
I
Average, 66%
Standing up, from sitting posture;
A 145 lb. 240 lb. 65%
B 144 lb. 259 lb. 80%
C 155 lb. 250 lb. 61%
D 155 lb. 290 lb. 87%
E 172 lb. 289 lb. 68%
F 186 lb. 414 lb. 122%
G 233 lb. 402 lb. 72%
II
Average, 79%
Standing and "jouncing"
Ax 139 lb. 332 lb. 139%
B 144 lb. 400 lb. 178%
C 155 lb. 440 lb. 184%
D 155 lb. 360 lb. 132%
F 186 lb. 625 lb. 236%
III
Average, 174%

FIG. 2.

The movement is indicated clearly by the photographs, Figs. 3 and 4, in which the two successive positions of the subject are shown by a double exposure of the negative. The records were made in precisely the same manner as for Experiment I, with the results as shown in Fig. 2, II. The letters, *A, B, C,* etc., refer to the same in-

dividuals as in I. The following is a sample "log-sheet" for this experiment, being the record for *B:*

(Name of subject.) Scale reading = 155 lb. (Date.)
Weight of chair = 11 lb.

Static weight of subject = 144 lb.

Scales set at 200 lb. . . . . Raised.
" " " 220 " . . . . . Raised.
" " " 240 " . . . . . Raised.
" " " 250 " . . . . . Raised.
" " " 260 " . . . . . Not raised.
" " " 260 " . . . . . Not raised.
" " " 260 " . . . . . Raised—third trial.
" " " 265 " . . . . . Raised—first trial.
" " " 270 " . . . . . Raised—first trial.
" " " 275 " . . . . . Not raised in three trials.

270 — 11 = 259 lb., total net effect, static + kinetic;
= 115 lb. increase, or 80 per cent.

It is interesting to note that this movement, which may readily occur in a theater, grand-stand, or other place where a large audience is gathered, gives distinctly higher values than are found in Experiment I, the average increase being nearly 80%, and no individual case, of the seven recorded, falling below 60 per cent.

III.—Jouncing.—In this experiment, the results of which are shown by III, Fig. 2, an attempt was made to get an approach to a maximum individual effect. The subject, standing on the scale platform, suddenly bent the knees and as quickly straightened them again, at the same time jerking the arms and shoulders downward to intensify the effort exerted. The instructions were to get as vigorous a movement as possible without letting the feet leave the scale platform. The vertical movement of the body did not probably exceed 2 or 3 in. The experiment is interesting and suggestive, but hardly of much importance, as the time during which the greatest effort acts is necessarily exceedingly short. The effort is more in the nature of a quick blow. Its bearing in relation to I and II will be discussed later.

Horizontal Effect.—I.—Some idea of the horizontal effort exerted in Experiment II (rising from a sitting posture) may be obtained by

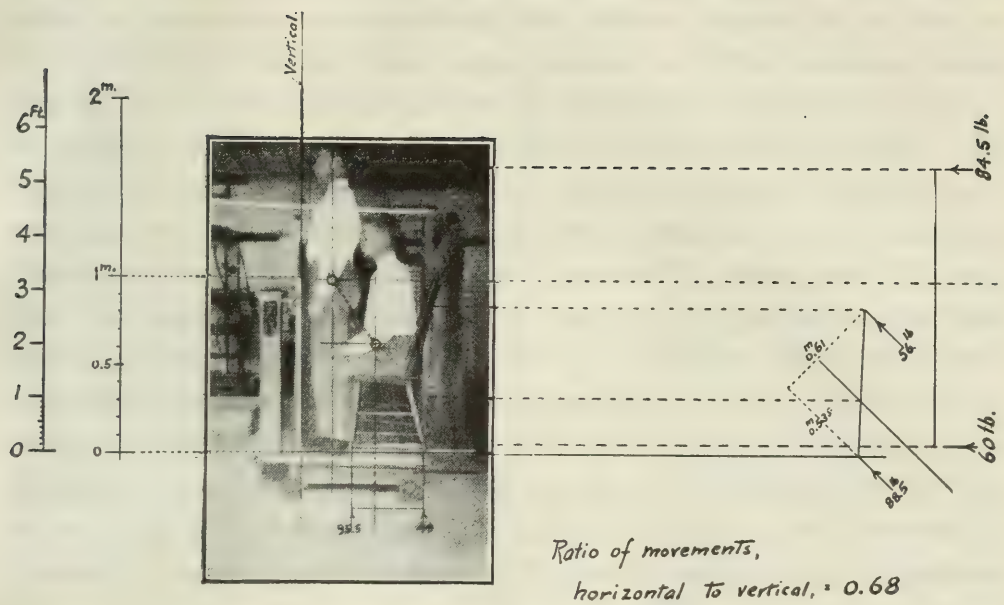

Vertical.
6 Ft.
5
4
3
2
1
0
2 m.
1 m.
0.5
0
84.5 lb.
60 lb.
Ratio of movements,
horizontal to vertical, = 0.68

FIG. 3.

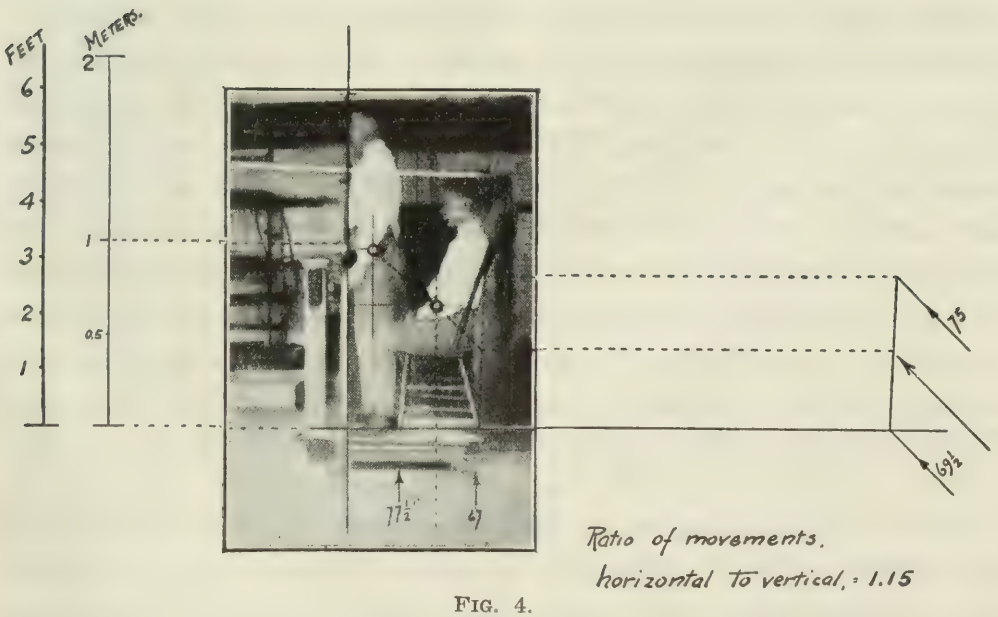

FEET
METERS.
6
5
4
3
2
1
2
1
0.5
75
69½
Ratio of movements.
horizontal to vertical, = 1.15

FIG. 4.

studying the movement of the center of gravity of the subject (Figs. 3 and 4). If the ratio of the movements, horizontal to vertical, is taken as measuring approximately the relative intensities of the respective forces, it gives a fairly good indication of the backward shove exerted by a man when he jumps up to cheer a spectacular play on the football field, or to shout welcome to his political favorite. In Fig. 3 the movement shown is from an erect sitting posture to a standing position; Fig. 4 shows a similar movement from a less alert sitting position. The first movement is probably more nearly typical than the second, but, in jumping up suddenly, as in Fig. 3, the final standing posture would probably have a somewhat greater forward inclination than is shown in the photograph, so that a ratio of horizontal to vertical motion of about 0.75 might fairly be assumed. Although this ratio of movements may not show the relative force values exactly, owing to complexities of motion not taken into account in the simple geometry of the figure, it is undoubtedly fairly indicative. On such a basis, taking the increase in vertical effect at about two-thirds of the static weight, it appears that, in rising suddenly from his seat, a man may exert a backward horizontal shove about one-half as great as his weight, say from 70 to 80 lb. for an ordinary man.

Figs. 5, 6, and 7 are introduced to show the method of determining the centers of gravity of the standing and sitting figures. For the standing figure (Figs. 3 and 4) the vertical centroidal line was readily drawn on the photograph by eye. The position of the horizontal centroidal line was then determined as in Fig. 5, by getting the reactions at head and heels on the two platform scales when the man was stretched out horizontally. A simple proportion between the two observed scale-weights then serves to fix the position of the line, and this is transferred to the photographs (Figs. 3 and 4) by the construction shown at the right of the figures. Similarly, for the sitting figure, a vertical centroidal line was obtained by the method shown in Fig. 6, and an inclined centroidal line as in Fig. 7, the angle of inclination of the latter being determined by the two measurements, 24 in. horizontal and 21 in. vertical, shown in the sketch. The intersection of these two lines, when properly transferred to the sitting figure in the photograph (geometrical construction on the right), locates the center of gravity of the seated man. The scale for the photographs was fixed by the two meter sticks seen clearly in Figs. 3 and 4, one laid

horizontally on the scale platform between the feet of the subject and the other standing vertically in the same plane. The vertical and horizontal components of the movement of the center of gravity were then measured directly on the photograph with this scale.

II.—Horizontal Forces Exerted by a Man Walking.—The motion of a man who is walking on a level floor, sidewalk, etc., at ordinary speed, is not perfectly uniform. The changes in velocity—whether acceleration or retardation—must be accompanied by some manifestation of horizontal force, this force urging the man forward when his velocity is being increased, and holding him back when it is being diminished. As these forces act on the man, his action (or reaction) on the floor must be equal and opposite to them; therefore, he will exert a backward horizontal push when his speed is increasing and a forward horizontal thrust when that speed is, for the moment, partly checked. The investigation which is summarized in Fig. 8 is an attempt to determine approximately what values these momentary forward and backward forces may have.

The twelve figures shown in the lower half of Fig. 8 were traced from a series of photographs, taken at intervals of $\frac{1}{12}$ sec., of an athlete walking.* The man walked directly in front of and parallel to a screen which was marked off by horizontal and vertical lines into squares of 5 cm. on a side. The subject was about 30 in. in front of this background, and the camera some 50 ft. from the subject, so that the distance interval covered in each interval of time might be determined with a close degree of approximation by counting the squares passed over by the man in each successive position. In making this count, the horizontal line, *AB,* was chosen as the base, as it passes very close to the center of gravity of the man for all positions, and the "reading" on the background squares was made at the intersection of this line with the buttocks of the naked figure. The total interval found in this way, from the first to the twelfth figure, was nearly 1.7 m., or about 5.5 ft., giving an average velocity of 6 ft. per sec. (a trifle more than 4 miles per hour) for the complete stride shown in the diagram. The average velocity during each interval ($\frac{1}{12}$ sec.) of

* This series of figures was traced from Plate 2 of Volume I of "Animal Locomotion," by Eadweard Muybridge (Philadelphia, 1887). This remarkable work, published in eleven great volumes, shows literally thousands of photographs of human beings and animals in great variety of motion. A smaller edition, containing a few of the series, much reduced in size, has been published under the title "The Human Figure in Motion." (London, Chapman and Hall, 1901.)

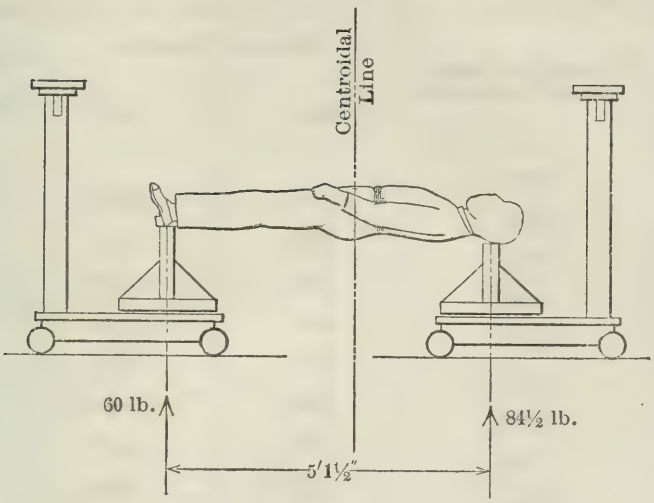


FIG. 5.

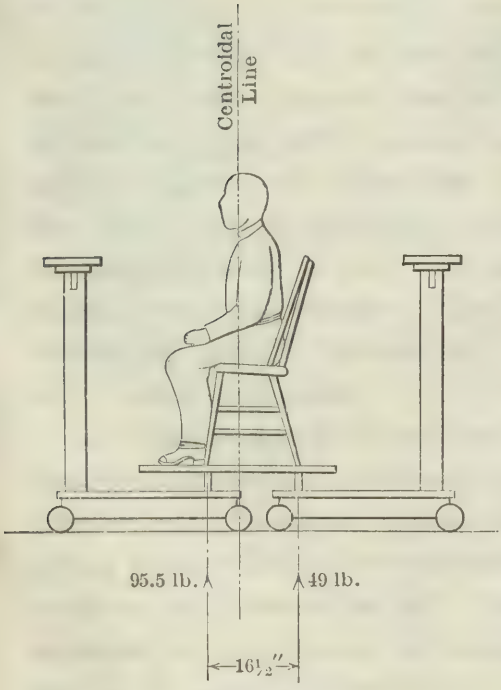


FIG. 6.

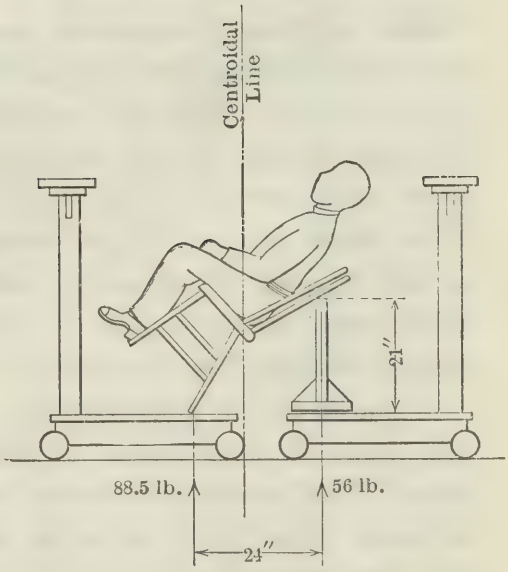


FIG. 7.

time may also be determined, and this has been done in the last column of Table 1.

TABLE 1.

Photographs.	Reading off background, in meters.	Distance Interval:		Velocity during interval, in feet per second.
		Meter.	Foot.	
1	0.18			
		0.18	0.59	7.1
2	0.36			
		0.14	0.46	5.5
3	0.50			
		0.12	0.39	4.7
4	0.62			
		0.15	0.49	5.9
5	0.77			
		0.15	0.49	5.9
6	0.92			
		0.20	0.65	7.8
7	1.12			
		0.16	0.53	6.4
8	1.28			
		0.14	0.46	5.5
9	1.42			
		0.13	0.43	5.2
10	1.55			
		0.13	0.43	5.2
11	1.68			
		0.17	0.56	6.7
12	1.85			

In interpreting and applying these results, the uncertainties attendant on their computation must be kept in mind; they give, however, an interesting and instructive indication of the way in which the velocity varies. Thus, the velocity is well below the average between Positions 3 and 4, well above between Positions 6 and 7 and well below again between Positions 9 and 10. This is shown graphically in the upper curve of Fig. 8, which gives the velocity-time curve drawn as indicated by the computed values, not exactly through the plotted points but "smoothed out" to give an even, continuous curve. The average velocity for the stride is shown by the heavy dotted line.

To measure the force exerted, it is necessary to know the acceleration, and this is obtained by differentiating graphically the velocity curve. This acceleration curve, or "first-derived" of the velocity curve, is shown immediately below the latter. The ordinates of the acceleration curve were obtained by actual geometric construction for the slope of the tangent to the velocity curve at a number of points. It is seen that the high and low points of this acceleration curve have values, plus and minus, respectively, of 16 or 18 ft. per sec. per sec., which would correspond to a horizontal accelerating or retarding force of

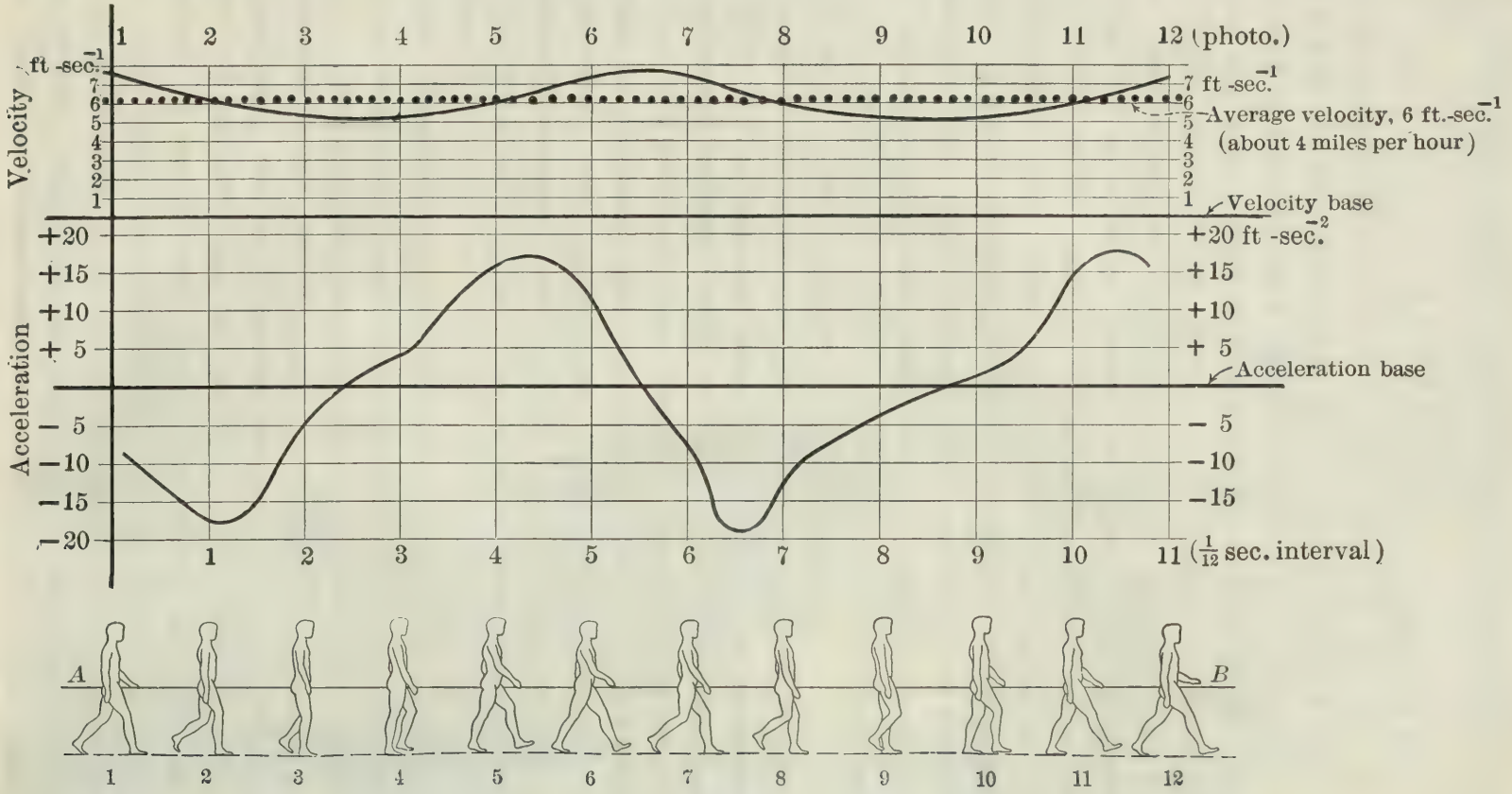

1 2 3 4 5 6 7 8 9 10 11 12 (photo.)
Velocity
ft -sec.$^{-1}$
7 6 5 4 3 2 1
7 ft -sec.$^{-1}$
Average velocity, 6 ft.-sec.$^{-1}$
(about 4 miles per hour)
Velocity base
Acceleration
+20 +15 +10 + 5
− 5 −10 −15 −20
+20 ft -sec.$^{-2}$
+15
+10
+ 5
Acceleration base
− 5
−10
−15
1 2 3 4 5 6 7 8 9 10 11 ($\frac{1}{12}$ sec. interval)
A
B
1 2 3 4 5 6 7 8 9 10 11 12

FIG. 8.

80 or 90 lb. for a man weighing about 160 lb.* These force values urge the man forward for the high points of the curve, and backward for the low points; note, in this connection, the backward shove or kick of the right foot in Positions 5 and 6 and of the left foot in Positions 11 and 12, and the forward thrust, retarding the body's motion, of the right foot in Positions 1 and 2 and of the left foot in Positions 7 and 8. Although these forces are impulsive in their nature and act for only a fraction of a second, there seems to be no doubt of their existence, and although the amount is not determinable exactly (varying as it must with different individuals and conditions), it may, apparently, equal or even exceed half the static weight of the man.

III.—Horizontal Forces Exerted by a Man Running Across a Bridge.—Approaching from a slightly different viewpoint this problem of horizontal force exerted, some experimental data were collected bearing on the lateral effect which may be produced on a bridge. At the time of a boat race, river pageant, or similar exhibition, a crowd of people will naturally gather on a bridge, ranging themselves along one side. As the spectacle moves under the bridge the entire crowd will cross to the other side (Fig. 9), and the motion is likely to be fairly rapid. What effect has it on the structure?

As in the previous cases, the experimenting was on an individual. The observer, with a stop-watch, noted the time of the subject as he turned from the rail on one side of the bridge and ran quickly to the other. This time included the start from rest at one rail and coming to rest again at the other. The suggestion was made that the subject imagine he was watching an exciting boat race, and did not want to miss any of it as the contestants pulled under the bridge. The tests were carried out on three bridges, in three different localities, the widths between rails being 17, 49½, and 68 ft., respectively, the second and third bridges having sidewalks on both sides. Three men took part as subjects, one a vigorous active man of sixty-seven, weighing 160 lb., another, thirty-nine years old, and weighing about 140 lb., and the third, twenty-six years old, and weighing 144 lb. The results are given in Table 2, and show, as might be expected, the higher average velocities for the wider bridges.

* Force (in pounds) = Mass × Acceleration, the mass being measured in "engineers' units" (1 unit = 32.2 lb.) and the acceleration in feet per second per second.

FIG. 9.—CROWD RUSHING FROM ONE SIDE OF A BRIDGE TO THE OTHER.

Remembering that these figures for the velocity are average values and not the maximum, we may obtain an approximation to the horizontal force exerted by the man, first as he started on his journey across the bridge and secondly when he stopped at the other side. Take, for example, the second bridge noted in Table 2 and the first experiment. The maximum speed may be safely assumed as about 12 ft. per sec.—more, probably, rather than less—and the "mass" of the moving man at 5 units ($160 \div g$). The familiar expression for kinetic energy, $\frac{1}{2} MV^2$, then yields,

$$\tfrac{1}{2} \times 5 \times 144 = 360 \text{ ft-lb. of kinetic energy,}$$

and this energy must be given to the man at the start of his trip and destroyed at the end. If this is accomplished in the space of 2 or 3 ft. ($\frac{1}{3}$ to $\frac{1}{4}$ sec.), as seems reasonable, the impulsive starting, or stopping, force has a value of about 150 lb., nearly equal to the weight of the man. If it takes a longer time to attain the maximum velocity from the standing start—say, about $\frac{1}{2}$ sec.—the effort may be extended over a distance of 5 or 6 ft., and will amount to perhaps 60 or 70 lb. This effort, of course, must be exerted both at the start and at the finish, as

TABLE 2.

Width of bridge, in feet.	Weight of subject, in pounds.	Time of running across bridge, in seconds.	Average velocity for one trip, in feet per second.
17	144	1.6 (down-stream) 1.4 (up-stream) 1.6 (down-stream) 1.6 (up-stream) Average.....	10.6 12.1 10.6 10.6 11 ft-sec.$^{-1}$
17	140	1.4 (up-stream) 1.6 (down-stream) 1.8 (up-stream) 1.6 (down-stream) Average.....	12.1 10.6 9.4 10.6 10.7 ft-sec.$^{-1}$
49.5	160	4.8 (up-stream) 4.4 (down-stream) Average.....	10.3 11.2 10.8 ft-sec.$^{-1}$
49.5	140	4.0 (up-stream) 4.4 (down-stream) Average.....	12.3 11.2 11.8 ft-sec.$^{-1}$
68	160	6.0 (up-stream) 5.4 (down-stream) Average.....	11.3 12.6 12 ft-sec.$^{-1}$
68	140	5.2 (up-stream) 4.8 (down-stream) Average.....	13.1 14.2 13.7 ft-sec.$^{-1}$

the start is from rest and the subject comes to rest again at the opposite rail. If, as may happen when he brings up sharply against the rail, the whole amount of kinetic energy is destroyed in a space of 6 in. or 1 ft., the effort will amount to several hundred pounds.

In applying these experimental results to determine the probable effect of crowds, several points must receive careful consideration. In the first place, the object of the whole inquiry is to get some sort of answer to the question: Against what loads, horizontal and vertical, should an engineer design a structure which is likely to have to carry a dense crowd of human beings? It is with respect to their bearing on this question that the experiments will be discussed.

Direct multiplication of individual effect by the probable number of units in the crowd is, of course, wrong. In the first place, the forces exerted are impulsive in their nature, being exerted for only a small, though finite and measurable, fraction of a second. In order to get the full effect of such impulsive efforts from a crowd of people, it is necessary to have perfect synchronism of motion in every individual, a condition practically out of the question. Further, the denser a throng of people, the more individual motion is restricted, so that in the more closely packed crowds, giving the higher static loads per square foot, the increase resulting from kinetic effect is much reduced. On the other hand, that the static load only shall operate, perfect quiescence must be secured, a condition quite as impossible in any crowd as that of absolute synchronism in movement; and the duty of the engineer is to provide in every case for the maximum possible load-effect to which his structure may be subjected, a duty which is immeasurably emphasized when the safety of human beings is at stake.

The first experiment—that of rising from a crouching position—although suggestive and interesting, hardly has a practical bearing. That particular form of motion could take place only in a sparse crowd, and even then would be highly improbable. Its main value lies in showing rather strikingly the importance of some consideration of kinetic effect.

The case of a man rising suddenly from his seat, however, is of considerable importance. No one who has watched a grand-stand full of enthusiastic football "fans" can doubt that a spectacular play may bring nine-tenths of them to their feet with such a close approxi-

mation to unanimity of motion that the total kinetic effect must be considerable. However, if the usual allowance of 3 sq. ft. per sitting is made, and each spectator is assumed to weigh 165 lb., or 55 lb. per sq. ft. over the whole structure, an increase of 65 or 70% (over the 55 lb. per sq. ft.) may be assumed without reaching the static value of 100 lb. per sq. ft. for which such a stand would probably be designed. Provision against horizontal effect, however, is not commonly made, and the importance of some such provision is illustrated by the experiment. To be on the safe side, a backward horizontal impulse of 70 or 80 lb. for each sitting might wisely be guarded against.

The "jouncing" movement, while it has a high kinetic intensity and is possible in a much denser throng than are the motions of Experiments I and II, is nevertheless of much shorter duration; the effect is that of a rather sharp, quick blow. On this account, practically perfect synchronism of movement is necessary to get the maximum effect, and this, of course, is quite impossible in any ordinary crowd.

The horizontal effect resulting from a man walking is probably not of general importance, except in the case of a large number of men marching in cadence, as a body of soldiers. The evil effect of this on bridges has been recognized for generations, and the tactical requirement of "breaking step" during the passage of a bridge by infantry is well known.* Mention might be made of the obvious slight variations in vertical load effect during the phases of a stride (in Fig. 8 it is seen that 4 and 9 are high positions and 1, 6, and 12 are low positions of the man's center of gravity), but these movements are small compared with those of Figs. 1 and 3.

The experiment of running across a bridge is perhaps rather more difficult of application to a crowd. If a line of men, each weighing 150 lb., was distributed at intervals of, say, 18 in. along the railing of a bridge, and, at a given signal, turned and ran to the other side, bringing up sharply against the opposite rail, the lateral force exerted might even exceed the usual allowance made for wind. Something of the kind, undoubtedly, may occur under certain conditions, but to what extent it should be guarded against is again a matter of individual judgment based on the exigencies of a particular problem. A further application of this experiment, however, might be mentioned.

* See quotation from Robert Stevenson, cited previously.

A wharf or pier, used for excursion boats, may collect a large number of people who enter from the land singly or in groups and come to rest on the structure. In coming to rest a horizontal force is exerted, that is, kinetic energy is destroyed, tending to push the wharf out into the water; and this force is applied after the manner of a succession of blows, of varying magnitude, but all in the same direction.

The same is true of any elevated platform, or structure, built for the accommodation of men and women, the entrance to which is restricted to only one line of movement. Such a structure is bound to receive shocks or impulsive horizontal forces in the manner indicated. That these effects are generally so small as to be of no importance is quite true; but that they may also on occasion reach considerable proportions, and especially that the cumulative effect may be serious, seems to be equally beyond question. It may be that a bridge, or pier, or platform may be fully capable of carrying all ordinary loads, even to a densely packed crowd of people, but some day it gives way under a much less (static) load. Is it not possible that the failure may be due to a peculiar combination of movements, on the part of the individuals of the crowd, timed and synchronized so that a force effect is produced on the structure far greater than that of the mere dead load?

In conclusion, it may be said that the evidence submitted hardly warrants any dogmatic statement as to what loads should be considered in providing for the possible effects of motion in crowds of people. The results of the experiments are not scientifically exact, nor are the applications definite or precise; but that some provision should be made against effects which are shown to be probable in an excited gathering, there can be no doubt. It is a matter for the individual engineer to determine, in solving his particular problem. The facts, albeit with some uncertainties and many qualifications, are presented as they appeared in the actual experiments described. They should be interpreted and used in the light of sound common sense, fair and impartial engineering judgment, and, above all, a due sense of the responsibility that rests on the architect or engineer for the safety of those who must use his construction.

In closing, the writer would express his obligations to the friends and colleagues whose cordial assistance is recorded in Fig. 2 and in the tables. He is particularly indebted to his father, to whose virile pencil Figs. 1 and 9 and part of Fig. 8 are due.

DISCUSSION

Mr. Hilder.

FRAZER C. HILDER, ASSOC. M. AM. SOC. C. E. (by letter).—This paper is a valuable addition to the scanty data on the loads to be expected on a structure crowded with moving people. It would seem to the writer that more extended experiments along these lines might well be undertaken to determine the horizontal component of the kinetic effect produced by a crowd moving in unison on a grand-stand or similar structure. A considerable horizontal force is evidently exerted by a crowd of "rooters," not only by springing to their feet at the crucial moment of an exciting game, as considered by Mr. Tilden, but by moving rhythmically thereafter so as to produce and continually accentuate a rocking motion in the structure. The writer has in mind an instance coming under his observation where a wooden stand was dangerously racked by a crowd of excited students swaying in unison, from one leg to the other, as they sang. While such ebullitions might well be suppressed in the interests of safety, it is evident that in spite of precautions such movement may occur, and that the possibility should be considered in the design of a grand-stand, particularly for a college.

While such an investigation would indicate the need of excessively heavy bracing, if an ordinary factor of safety were used, it would seem that a structure of the class indicated might well be designed to resist the possible rocking or marching of a crowd, with a small factor of safety, as compared with that calculated for static loads.

Mr. Quimby.

HENRY H. QUIMBY, M. AM. SOC. C. E. (by letter).—Any suggestion of new forces to be provided for in the design of structures will awaken the interest of a structural engineer, and, therefore, the experiments described in this paper command attention. The statement that a man in rising suddenly from a seat may exert a horizontal effect or force equal to one-half his weight, is somewhat startling; and that a grand-stand should be figured for a horizontal force equal to one-half of all its vertical live load is a new proposition, which, if true, is important.

In order to test the proposition, the writer rigged up a means of measuring horizontal impulses by attaching adjustably loaded brackets to a light swinging platform; he also experimented with the swing disconnected from the loads, in order to observe the amount of the induced lateral movement.

The greatest backward horizontal force produced by a sudden rising from the seat with the consequent shifting forward of the center of gravity of the body, appeared to be 7% of the body weight, and the backward swing of the unrestrained platform indicated only 5% when the weight was multiplied by the tangent of the angle of incli-

Mr. Quimby.

nation of the suspenders. The latter is probably not at all reliable, because the amount of the lateral swing is not likely to be in direct proportion to the length of the suspenders, and, without doubt, the backward swing was always interrupted by the forward impulse.

It was immediately observed that the re-acting forward force was much greater than the rearward force. The unrestrained platform—disconnected from the apparatus and free to swing—would first swing backward about 4 in., and then about 8 in. forward of the neutral point. When it was restrained, the weight lifted by the second or return impulse was about twice as great as that lifted by the first or backward impulse, that is, about 14% of the vertical load.

These measurements seem to indicate that there is something wrong with the author's method of computing the horizontal effect. The error is probably in the assumption that the horizontal movement is as rapid as the vertical one. A study of the movement shows that either the center of gravity is thrown forward before rising, or the feet are drawn under the body and, after rising, the body steps forward, for the rise could not be made in the inclined direction without a "purchase" to the rear to push against. Speed of movement is essential to the development of kinetic energy.

The rearward impulse is succeeded so quickly by the forward one that the bracing of the structure has hardly time to feel it, and its energy must be largely absorbed by the inertia of the structure if the dead weight is at all considerable, and this is likely to be the case always. The platform used in the experiment described was very light—perhaps one-fifth of the live load—as probably was that of the scale used by the author in his work. Absorption of energy of a transitory force reduces the effect and the attending stress, as illustrated by the old magic of enduring a sledge-hammer blow on one's chest by interposing an anvil. It is suggested that the author repeat his rising experiment on the scale with its platform loaded with a dead weight equal to the live load, and report the result.

The fact that the second or forward impulse is greater than the rearward one, is probably due to the arrest of the movement being more sudden than its start. The energy expended in the movement must be divided between accelerating it and retarding it, but the kinetic effect will be proportioned to the quickness of each stage.

The stress, or the effect of stress, in a structure is not directly proportional to the speed of application of loads—it is not necessarily increased by so-called impact. Within limits, the rapid application of a load to almost any material will find a higher resistance than a slow application, and if the dead weight be equal to, say, the live load, much of any kinetic effect of sudden loading will be lost in the mass before reaching the bracing.

Mr. Quimby.

The effect of a crowd running across a bridge and against the railing is a more menacing force, and at least one case is recalled where a footbridge over a stream collapsed under a shifting crowd of spectators at a boat race. A well-designed bridge railing, however, will be competent to resist the pressure of a crowd, for if the strength of the rail and the posts be computed for 50 lb. per lin. ft. applied at the rail, the usual allowed unit stress will provide ample margin for a push against it. The kinetic effect of a crowd hurrying across the bridge would be spent in the floor and not on the railing, for only one rank of the runners could fetch up against the rail—at least, at one moment—and a bridge wide enough to permit a shifting crowd to get up any speed must have considerable mass or inertia, and is likely also to be stiff as a horizontal beam.

It would seem, therefore, that we hardly need to revise our practice in the design of structures to care for a new kind of impact. The suggestions in the paper, however, should be useful in directing attention to the necessity for intelligent design in the construction of all parapet railings, but the most formidable impact that railings are subject to nowadays seems to be that of straying trolley cars and automobiles, one of the former and half a dozen of the latter having come under the writer's notice recently, and engineers cannot well be expected to design for such contingencies.

Mr. Coombs.

R. D. Coombs, M. Am. Soc. C. E.—It seems to the speaker that the general application of the results of the experiments to specifications, by the addition of loading requirements, would be an unnecessary burden to structures which are calculated for wind load. Such structures, particularly if designed for the venerable loading of 30 lb. per sq. ft., would have a strength more than sufficient to care for the lateral stresses due to the movement of the live load. In fact, the calculated wind load would not occur while the structure was occupied, as the load presupposes a very violent wind velocity. Further, the usual so-called "wind loads" really include a vibration, or lateral impact loading.

Though some special provision for lateral and vertical impact forces may be proper for light temporary grand-stands, etc., which may not be designed for the usual wind loads, the speaker believes any such addition entirely unnecessary for first-class permanent work.

Mr. French.

J. B. French, M. Am. Soc. C. E.—The author's experiments are of interest, and merit discussion, but, as has been pointed out, the effect of these forces on structures is very largely dependent on the general character and dimensions of the structure, independent of its strength. If the intervals of time between the successive applications of even a very small force correspond to the rate of vibration

Mr. French. of the structure or structural member, the effect will be cumulative and may assume serious proportions.

The most extensive experiments on structures to determine the kinetic effect, as distinguished from the static effect, of moving loads, are undoubtedly those made by the American Railway Engineering Association by running trains across bridges, the length, weight, and speed of the trains, and the length of the bridges all being varied within considerable limits; and, in these experiments, it appeared to be demonstrated that the kinetic effect, or impact produced, depended largely on the relation of the span length to the speed and length of the train, and that the running of trains over bridges at a speed of less than 10 miles per hour produces no appreciable effect beyond that which would be produced by the train standing still.

The effect of the movement of crowds over bridges would certainly be dependent on similar laws, and it does not seem possible that a bridge designed to carry the automobile and heavy trucking traffic of to-day, and constructed with the solid floors which are now so common, could be appreciably affected by the dynamic effect of crowds of people.

In the case, however, of grand-stands filled with spectators of a football or similar game appealing to their enthusiasm, all might arise at one time and engage in regularly timed cheering or stamping, thereby, undoubtedly, producing a considerable effect, and this should be taken into account in the design of such structures. Even in this case, however, it is evident that the effect would be more dependent on the rigidity of the structure, or what might possibly be called its "vibratory pitch," than on its strength under static loads, or the weight of the spectators.

Mr. Tilden. C. J. Tilden, M. Am. Soc. C. E. (by letter).—Mr. Hilder and Mr. French raise a question which is somewhat outside the scope of the paper, namely, that of vibrations set up in a structure by the movements of its living load. This, of course, is one possible result of the kinetic effect of crowds, and if the swaying motion of the crowd is in unison with the vibration period of the structure, disaster may follow. This phase of the question, however, has to do more with the building than with the load on it, and, therefore, was intentionally omitted from the original investigation. Nevertheless, the writer is glad to have the point raised, and does not by any means underestimate its importance.

The results reported by Mr. Quimby of his experiments with the swinging platform are not at all convincing. The existence of a backward-shoving force is clearly shown by the sudden backward swing of the platform, but to measure it by calling it equal to the force needed to hold the platform the same degree out of plumb, is surely a serious error. The immediate force-effect of a moving body on a freely swinging structure is very different from that of the same body,

Mr. Tilden.

moving in the same manner, on a practically rigid structure. The writer still believes that the maximum possible backward push on a grand-stand, or similar structure, is much more likely to be 50%, or more, of the static weight than the 7% suggested by Mr. Quimby. Referring to the writer's computation, Mr. Quimby says: "The error is probably in the assumption that the horizontal movement is as rapid as the vertical one."

It is hard to see just where this "error" lies, as the two movements are made in precisely the same time, as a glance at Figs. 1 and 2, with their accompanying explanations, will show.

The absorption of a large part of these impulsive forces by "the inertia of the structure" is, of course, quite probable where the forces are isolated or result from the sudden movement of small and separated groups. It is when the movement is so general as to be practically unanimous that the resulting force on the entire structure may reach serious proportions. The experiment of interposing, between the man's feet and the scale platform, a mass nearly equal to the weight of the man, was tried by the writer, and the results reported in the last paragraph of page 2110 and the lines A_x of Fig. 2.

Mr. French is quite right in saying that a heavy modern bridge, with solid floor, could hardly be seriously affected by the movements of a crowd of people. Unfortunately, however, such bridges are a very small part of all the bridges in use. It is not the "first-class permanent work" (to quote Mr. Coombs' expression) which needs attention along the lines suggested by the paper, but the much more common, more or less flimsy structures, often hastily built, and sometimes without proper supervision.

The writer believes that a general increase in loading requirements—possibly even to the extent of 50 or 60%—for structures likely to carry people would be wise and proper. Such a ruling would not materially affect first-class design, but would decidedly benefit the lighter classes of work. It is certainly better to be safe than to be economical, where human lives are concerned.

Emphasis is lent to this view by the reports of accidents of various kinds which frequently appear in the press. Two recent cases are commented on briefly:* The first of these was the accident at Lawrence, Mass., on June 30th, when a dozen boys, from 9 to 15 years of age, lost their lives through the failure of a narrow, wooden walk leading to a municipal bath-house. The Associated Press report said "the boys were jumping up and down as they shouted to the keeper to open up." The second instance was that of the second floor piazza of a tenement house at Brookline, Mass., which collapsed under the weight (plus kinetic effect?) of fourteen children, three of whom were badly injured.

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* *Engineering News*, July 10th, 1913, p. 90.

Mr. Tilden.

The recent failure of the floor of an amusement pier at Long Beach, Cal., resulting in the loss of 35 or 40 lives, was fully reported in the daily and engineering press. It is simply another instance in which it is not hard to imagine the kinetic effect, or impact, of the densely packed crowd as being at least a contributory cause of the collapse. At the ceremony of notifying Governor Marshall of his selection as Democratic candidate for Vice-President, which took place in Indianapolis in August, 1912, a grand-stand containing several hundred people gave way. Fortunately, no lives were lost, and the incident was passed over briefly by the press; but, here again, one may imagine movements in such a crowd, inspired by political enthusiasm, which might have much greater effect than the mere dead weight.

These are simply reports of facts, serious facts, involving the loss of many lives and the endangering of many more. It would be an easy matter to multiply instances, for scarcely a month passes that some accident of the kind is not reported. It is difficult, of course, to establish any direct and unquestioned connection between them and the subject of the paper. We naturally look for weakness in the structure rather than for excess in the load; but a more general recognition of the possible effects of motion in crowds would, the writer believes, result in safer and more rational design. Even practical builders, who might not be able to compute stresses and strains, would quickly "sense" the new requirements and build accordingly.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1280

BIBLIOGRAPHY ON VALUATION OF PUBLIC UTILITIES.

Prepared in the Library of the Society,
By the Library Force, May 15th, 1912,
For the Use of the Special Committee on
Valuation of Public Utilities.

This bibliography has been brought up to July 16th, 1913, and is here published in the belief that it will be useful to members of the Society.

The References are Arranged Under the Following Headings:

GENERAL	STEAM POWER
ELECTRIC LIGHT AND POWER	STREET AND INTERURBAN RAILWAYS
GAS-WORKS	TELEGRAPH AND TELEPHONE
MINING	WATER POWER
RAILROADS	WATER-WORKS

Note.—Unverified references to publications which are not in the Library of the Society are placed at the end of each Division of the List.

Articles marked with an asterisk are published by State Commissions, or contain extracts from their reports.

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METHODS OF DETERMINING LIFE OF PUBLIC UTILITIES. *Engineering and Contracting*, v. 38, p. 448 (Oct. 23, 1912). (Based on a paper by Halford Erickson on depreciation, read before the Central States Water Works Association.)

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COMMENTS ON FIXED COSTS IN INDUSTRIAL POWER PLANTS; by John C. Parker. *Proceedings*, American Institute of Electrical Engineers, v. 30, p. 469 (March 30, 1911). (Two and one-half pages on depreciation.)

COMMERCIAL DEPRECIATION IN ELECTRIC PLANTS. *Street Railway Bulletin*, v. 7, p. 431 (Aug., 1908). (States that allowance should be made for machinery out of date in addition to allowance of 10% for ordinary wear and tear.)

COMMERCIAL DEPRECIATION IN ELECTRIC PLANTS; by Judson H. Boughton. *Public Service*, v. 5, p. 7 (July, 1908). (One-half page.)

DEPRECIATION; by C. N. Duffy. *Electrical World*, v. 51, p. 217 (Feb. 1, 1908). (The treatment of depreciation is confined to broad general questions briefly touched upon as applicable to electric lighting; abstract of paper read before the Northwestern Electrical Association.)

——*Electric Railway Review*, v. 19, p. 83 (Jan. 18, 1908).

——*Electrical Review* (London), v. 63, p. 374 (Sept. 4, 1908).

——*Street Railway Journal*, v. 31, p. 169 (Feb. 1, 1908).

——Editorial. Depreciation. *Electrical World*, v. 51 p. 207 (Feb. 1, 1908).

DEPRECIATION; by Robert Hammond. *Journal*, Institution of Electrical Engineers, v. 39, p. 270 (1907). (The question of depreciation in all its bearings as applicable to electricity supply undertakings.)

——Abstracts. *Electrician*, v. 59, p. 51 (April 26, 1907); *Electrical Review* (London), v. 60, p. 744 (May 3, 1907); *Electric Railway Review*, v. 17, p. 716 (June 1, 1907); *Progressive Age*, v. 25, p. 305 (June 1, 1907); *Engineering Magazine*, v. 33, p. 636 (July, 1907); *Engineering Record*, v. 55, p. 703 (June 15, 1907); *Electrical Review* (Chicago), v. 50, p. 828 (May 25, 1907); *Municipal Journal* (London), v. 16, pp. 411, 435 (May 10, 17, 1907); *Street Railway Bulletin*, v. 6, p. 382 (June, 1907); *Tramway and Railway World*, v. 21, p. 497 (June 6, 1907).

——Editorials and discussions. *Electrician*, v. 59, pp. 100, 103 (May 3, 1907); *Engineering Record*, v. 55, p. 703 (June 15, 1907).

DEPRECIATION. (Letter); by S. Fred Smith. *Electrical World*, v. 54, p. 489 (Aug. 26, 1909). (Depreciation of property of electrical corporations; general.)

ELECTRIC LIGHT AND POWER—GENERAL—(*Continued*).

DEPRECIATION ACCOUNTING FOR SMALL COMPANIES; by George E. Claflin. National Electric Light Association, Thirty-second Convention, 1909, Papers, Reports and Discussions, v. 3, p. 165. (Classification for depreciation of electrical works; tangible property; wear and tear; obsolescence; inadequacy; extraordinary casualties.)

——Abstract. *Electric Railway Journal,* v. 33, p. 1078 (June 12, 1909). (Very brief.)

THE DEPRECIATION AND MAINTENANCE OF ELECTRICAL EQUIPMENT; by George W. Cravens. *Electrical Review* (New York), v. 56, p. 853 (April 23, 1910). (Considers the different methods of accounting in use and advocates the sliding scale method; four pages.)

DEPRECIATION AND REPAIRS. (Editorial.) *Electrical Review and Western Electrician,* v. 53, p. 807 (Nov. 28, 1908). (Allowance made for electric lighting plants for annual depreciation and repairs.)

DEPRECIATION AND RESERVE FUNDS OF ELECTRICAL PROPERTIES; by William B. Jackson. *Journal,* Western Society of Engineers, v. 15, p. 587 (September, 1910). (Discusses methods of estimating the amount to be charged for depreciation and reserve fund and how the principle should be applied; thirty-two pages.)

——Abstracts. *Engineering-Contracting,* v. 33, p. 487 (May 25, 1910); *Electric Railway Journal,* v. 35, p. 903 (May 21, 1910).

DEPRECIATION AND RESERVES FOR ANTIQUATION AND OBSOLESCENCE FROM an Engineering Standpoint; by C. H. Yeaman. *Electrician,* v. 59, p. 475 (July 5, 1907). (Contains table of estimated life of electrical appliances for loan purposes.)

——*Electrical Engineer* (London), v. 40, p. 46 (July 12, 1907).

——*Electrical Review* (London), v. 61, p. 44 (July 12, 1907).

DEPRECIATION AS RELATED TO ELECTRICAL PROPERTIES; by Henry Floy. *Proceedings,* American Institute of Electrical Engineers, v. 30, p. 1267 (1911). (A long article, subdivided under application of terms, classes of depreciation, absolute and theoretical depreciation, depreciation accounts or reserve funds, 50% method, depreciation of contingent percentages and summary and conclusions.)

——Abstracts. *Electric Railway Journal,* v. 38, p. 21 (July 1, 1911); Depreciation. *Engineering-Contracting,* v. 36, p. 359 (Oct. 4, 1911); Notes on Depreciation. *Engineering Record,* v. 64, p. 282 (Sept. 2, 1911).

——Comments. Absolute and Theoretical Depreciation. *Engineering Record,* v. 64, p. 333 (Sept. 16, 1911); Depreciation; by H. C. D. Nutting. *Electrical World,* v. 58, p. 323 (Aug. 5, 1911).

DEPRECIATION OF COMPANIES' ASSETS. (Letter.) *Electrician,* v. 59, p. 146 (May 10, 1907). (On depreciation of electric light plants; very brief.)

DEPRECIATION OF ELECTRIC LIGHT PLANTS; by Alexander C. Humphreys. *Municipality,* v. 8, p. 72 (March, 1908). (The elements of obsolescence, inadequacy and actual decay.)

DEPRECIATION OF ELECTRIC LIGHT PLANTS; by Robert Hammond. *Municipality,* v. 8, p. 69 (March, 1908). (An attempt to secure data on actual depreciation, rather than methods used in appraisal.)

DEPRECIATION OF ELECTRIC LIGHT PLANTS; by William H. Bryan. *Municipality,* v. 8, p. 74 (March, 1908). (Brief abstract of paper read before the Engineers' Club of St. Louis.)

THE DEPRECIATION OF ELECTRICAL PROPERTIES; by G. W. Bissell. *Electrical Age,* v. 36, p. 459 (June, 1906). (The allowance that should be made for depreciation.)

DEPRECIATION OF POWER-PLANT EQUIPMENT. (Letter); by Everard Brown. *Electrical World,* v. 60, p. 268 (Aug. 3, 1912). (On decrepitude and obsolescence of machinery in electrical power plants.)

DEPRECIATION OF POWER PLANT EQUIPMENT; by F. H. Neely. *Power,* v. 30, p. 1028 (June 8, 1909). (Concerning depreciation in private and municipal plants and provision which should be made against it.)

DEPRECIATION ON ELECTRIC LIGHT AND POWER PLANTS. *Electric Railway Journal,* v. 40, p. 60 (July 13, 1912). (Analysis of depreciation on different elements of physical property; brief.)

ELECTRIC LIGHTING RATES AND DEPRECIATION; by H. H. Crowell. *Municipal Journal and Engineer,* v. 23, p. 698 (Dec. 18, 1907). (Table of estimated life of apparatus, depreciation due to wear, obsolescence and inadequacy.)

ELECTRICAL UNDERTAKINGS AND THE LAW OF RATING. (Serial.) *Electrical Review* (London), v. 66, p. 84 (Jan. 21, 1910). (The first part discusses the rating of electric light and power companies.)

ELECTRIC LIGHT AND POWER—GENERAL—(*Continued*).

ELEMENTS AFFECTING THE FAIR VALUATION OF PLANT AND PROPERTY; by W. F. Wells. National Electric Light Association, Thirty-fourth Convention, 1911, Papers, Reports and Discussions, v. 1, p. 271. (Analysis of valuation classification of electrical properties.)

ESTIMATING THE COST OF AN ELECTRIC PLANT. *Journal,* Franklin Institute, v. 165, p. 397 (May, 1908). (Gives years of life as estimated by different engineers for various parts of the plant.)

MAKING RATES FOR ELECTRIC PLANTS; by Halford Erickson. *Public Service Regulation,* v. 1, p. 579 (Sept., 1912). (Principles of valuation; going value, depreciation, operating expenses, rates, effect of demand on cost, etc.)

THE OBSOLESCENCE OF ELECTRIC LIGHTING PLANT; by F. Fernie. *Electrical Review* (London), v. 63, p. 516 (Sept. 25, 1908). (Discusses rate of depreciation and necessity for an insurance fund.)

RATE-MAKING FOR PUBLIC UTILITIES; by Halford Erickson. *Electric Railway Journal,* v. 33, p. 775 (April 24, 1909). (Relation between investment and output of electrical plants; paper read before the Wisconsin Electric and Interurban Railway Association.)

RATE REGULATION OF ELECTRIC POWER; by S. S. Wyer. *Cassier's Magazine,* v. 35, p. 410 (Jan., 1909). (Considers replacement value and depreciation.)

RATES AND RATE MAKING; by John F. Druar. *Journal,* Association of Engineering Societies, v. 50, p. 221 (May, 1913). (Discusses the valuation of a combined electrical and gas property to determine the legitimate capital, on which capital a certain return should be received.)

STANDARD HANDBOOK FOR ELECTRICAL ENGINEERS, p. 668. Edition 3. McGraw-Hill Book Co., New York, 1910. (Contains brief data on cost and depreciation of electric plants.)

UNIFORM SYSTEM OF ACCOUNTING. (Letter); by F. E. Haskell. *Electrical World,* v. 53, p. 928 (April 15, 1909). (Rule adopted to provide for monthly charge to cover wear and tear, obsolescence and inadequacy, etc.)

UPKEEP CHARGES ON LARGE ELECTRIC GENERATING SETS; by Robert J. Burstall. *Electrical Engineer* (London), v. 39, p. 866 (June 21, 1907). (Paper read before the Engineering Conference, Institution of Civil Engineers; allowance for repairs and renewals.)

——*Engineering,* v. 83, p. 834 (June 21, 1907).

VALUATION OF ELECTRIC PLANTS. (Editorial.) *Engineering Record,* v. 58, p. 365 (Oct. 3, 1908). (One and one-half columns.)

ELECTRIC LIGHT AND POWER—SPECIAL CASES.

Aberdeen, Scotland.

ABERDEEN AND DEPRECIATION. *Municipal Journal* (London), v. 12, p. 943 (Oct. 23, 1903). (Comparison of allowance for depreciation of electrical plants at Aberdeen, Glasgow, and Bolton.)

Beloit, Wis.

***CITY OF BELOIT VS. BELOIT WATER, GAS AND ELECTRIC COMPANY;** Decided July 17, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 216. Madison, Wis., 1912. (Details of the valuation of the power plant and going value.)

FIXING NORMAL OPERATING COSTS; by Frank A. Newton. *Engineering Record,* v. 65, p. 258 (March 9, 1912). (Comments on the decision of the Wisconsin Railroad Commission in the case of the City of Beloit *vs.* Beloit Water, Gas & Electric Co.)

Boonville, N. Y.

AMORTIZATION RULE OF THE NEW YORK PUBLIC SERVICE COMMISSION OF the Second District. *Electrical World,* v. 54, p. 1162 (Nov. 11, 1909). (Computation of amortization of property of the Board of Light Commissioners of Boonville, N. Y.)

Bristol, England.

REPORT BY SIR WILLIAM PREESE ON PROBABLE LIFE OF PLANT AT BRISTOL. *Electrician,* v. 57, p. 704 (Aug. 17, 1906). (Details of estimated life of electrical plants; figures given for various items are those used by L. R. Dicksee in his report.)

——Editorial. Depreciation. *Electrician,* v. 57, p. 702 (Aug. 17, 1906).

Burkhardt Milling & Electric Power Co.

***E. G. ROSS ET AL. VS. BURKHARDT MILLING AND ELECTRIC POWER COMPANY;** Decided April 8, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 139. Madison, Wis., 1911. (The value of property and the method of determining values are discussed.)

ELECTRIC LIGHT AND POWER—SPECIAL CASES—(*Continued*).

California.

*UNIFORM CLASSIFICATION OF ACCOUNTS FOR ELECTRIC CORPORATIONS prescribed by the Railroad Commission of the State of California; Adopted Oct. 23, 1912, Effective Jan. 1, 1913. Sacramento, 1912.

Cardiff, Wales.

DEPRECIATION: INTERESTING REPORT FROM CARDIFF. *Municipal Journal* (London), v. 16, p. 1083 (Dec. 20, 1907). (Allowance for depreciation considered to represent fair wear and tear.)

DEPRECIATION OF CARDIFF ELECTRIC TRAMWAY AND LIGHTING UNDERtakings. *Electric Railway Review,* v. 19, p. 16 (Jan. 4, 1908). (Details of rates of depreciation of equipment are given.)

Cashton, Wis.

*IN RE DETERMINING AND FIXING JUST COMPENSATION TO BE PAID TO THE Cashton Light and Power Company by the Village of Cashton for the Taking of the Property of the Said Company Actually Used and Useful for the Convenience of the Public in Accordance With the Provisions of Chapter 499, Laws of 1907; Submitted Oct. 14, 1908, Decided Nov. 28, 1908. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 3, p. 67. Madison, Wis., 1910. (Discusses going value of public utility plants.)

Chippewa Falls, Wis.

*T. J. CUNNINGHAM ET AL. VS. CHIPPEWA FALLS WATER WORKS AND Lighting Company; In Re Investigation by the Railroad Commission of Wisconsin of Rates Charged by the Chippewa Falls Water Works and Lighting Company; In Re Valuation of the Property of the Chippewa Falls Water Works and Lighting Company; Decided June 14, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 302. Madison, Wis., 1911. (Contains data on the value of the electric plant.)

Chippewa Valley Ry., Light & Power Co.

*IN RE APPLICATION OF THE CHIPPEWA VALLEY RAILWAY, LIGHT AND Power Company for Authority to Change its Rates; Submitted Feb. 19, 1908, Decided Mar. 18, 1908. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 2, p. 311. Madison, Wis., 1909. (Refers to valuation of electric plant.)

Darlington, Wis.

*IN RE APPLICATION OF THE DARLINGTON ELECTRIC LIGHT AND WATER Power Company for Power to Increase Rates; In Re Darlington Electric Light and Water Power Company, Valuation of Property; Submitted Sept. 2, 1909, Decided June 17, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 397. Madison, Wis., 1911.

District of Columbia.

*UNIFORM SYSTEM OF ACCOUNTS FOR GAS CORPORATIONS AND ELECTRIC Corporations in the District of Columbia as Prescribed by the Interstate Commerce Commission, pp. 29, 47, 55, 67. Washington, 1909. (Provision for amortization of plant, which includes monthly charges of the amount estimated to be necessary to cover wear, tear and obsolescence.)

Dodgeville, Wis.

*CITY OF DODGEVILLE VS. DODGEVILLE ELECTRIC LIGHT AND POWER COMpany; Submitted May 4, 1908, Decided June 2, 1908. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 2, p. 392. Madison, Wis., 1909. (Data relating to valuation of plant.)

Edinburgh, Scotland.

DEPRECIATION, ETC., ON ELECTRICITY SUPPLY UNDERTAKINGS. *Electrician,* v. 57, pp. 231, 350 (May 25, June 15, 1906). (Report on present condition of electric light and machinery plant of the Edinburgh Corporation.)

Fareham, England.

DEPRECIATION. (Editorial.) *Electrician,* v. 62, p. 709 (Feb. 19, 1909). (Depreciation in connection with a loan for an electrical plant at Fareham, England.)

Greenwood, Miss.

REPORT OF THE APPRAISERS SELECTED TO ESTIMATE THE VALUE OF property of the Greenwood Light and Water Company to the City of Greenwood and the Greenwood Light and Water Co., March 22, 1904. Greenwood, Miss., 1904. (Eight pages.)

Groton, Mass.

A VALUABLE MUNICIPAL RATE DECISION. (Editorial.) *Engineering Record,* v. 66, p. 2 (July 6, 1912). (Comments on decision by the Massachusetts Gas and Electric Light Commission on electrical rates at Groton, Mass; very brief.)

ELECTRIC LIGHT AND POWER—SPECIAL CASES—(*Continued*).

Kaukauma, Wis.

*IN RE DETERMINING AND FIXING THE JUST COMPENSATION TO BE PAID to the Kaukauma Gas, Electric Light and Power Company by the City of Kaukauma; Submitted Feb. 6, 1911, Decided Dec. 26, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 8, p. 409. Madison, Wis., 1912. (Physical value and going value of the property and value of the water-power lease.)

La Crosse. Wis.

*IN RE APPLICATION OF THE LA CROSSE GAS AND ELECTRIC COMPANY FOR Authority to Increase Its Rates; Decided Nov. 17, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 8, pp. 138, 156, 170, 179, 202, 224. Madison, Wis., 1912. (Physical value of plants, original cost and effect of allowance of going value.)

*IN RE APPLICATION OF THE LA CROSSE GAS AND ELECTRIC COMPANY FOR Authority to Increase Rates; Submitted Aug. 16, 1907, Decided Sept. 19, 1907. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 2, p. 3. Madison, Wis., 1909. (Gives method of estimating cost of plant before physical examination can be made.)

*THE MEANING OF "ACTUAL STATION OPERATING COSTS." *Engineering Record,* v. 65, p. 191 (Feb. 17, 1912). (Decision of the Railroad Commission of Wisconsin in the case of the La Crosse Gas & Electric Co.)

Madison, Wis.

RATE MAKING FOR PUBLIC UTILITIES, THE MADISON CASE; by Percy H. Thomas. *Electric Journal,* v. 7, p. 560 (July, 1910). (Discusses the decision of the Railroad Commission of Wisconsin in the case of the State Journal Printing Co. *vs.* the Madison Gas & Electric Co., rendered March 8, 1910.)

Manitowoc, Wis.

*CITY OF MANITOWOC VS. MANITOWOC ELECTRIC LIGHT COMPANY; Submitted Sept. 30, 1908, Decided June 14, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 361. Madison, Wis., 1911. (A tentative valuation of the physical property of the respondent was made; the income accounts and operating expenses for a term of years are analyzed.)

Marinette, Wis. See Menominee, Wis.

Marquette, Mich.

A STUDY IN CENTRAL-STATION FINANCES AND OPERATION FROM MARQUETTE, Mich. *Electrical World,* v. 53, p. 403 (Feb. 11, 1909). (Gives estimates of depreciation for an electric light and power plant.)

Massachusetts.

DISTRIBUTION COSTS IN SEVEN CENTRAL-STATION SYSTEMS. *Electrical World,* v. 52, p. 1014 (Nov. 7, 1908). (Figures deduced from returns to the Massachusetts Board of Gas and Electric Light Commissioners.)

Menominee, Wis.

*IN RE VALUATION OF ELECTRIC LIGHT PLANT OF MENOMINEE AND MARInette Light and Traction Company; In Re Application of Menominee and Marinette Light and Traction Company for Authority to Equalize Rates; In Re Menominee and Marinette Light and Traction Company, Investigation of Rates on Motion of the Commission; Decided Aug. 3, 1909. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 3, p. 778. Madison, Wis., 1910. (Data on valuation of electric light and power plant.)

Merrill Ry. & Lighting Co.

*IN RE APPLICATION OF THE MERRILL RAILWAY AND LIGHTING COMPANY for Authority to Change Its Rates for Electric Lighting; Submitted Sept. 17, 1907, Decided Dec. 10, 1907. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 2, p. 148. Madison, Wis., 1909. (Discusses value of the plant, including water power and dam, electric light plant, and railway plant.)

Minneapolis, Minn.

ELECTRIC RATES FOR MINNEAPOLIS, A LONG CONTROVERSY OVER BASING Rates on Expert Analysis or Unreasonable Comparisons; by William G. Deacon. *Public Service,* v. 5, p. 107 (Oct., 1908). (Contains very brief data on valuation.)

MINNEAPOLIS LIGHT AND POWER RATES. *Electrical World,* v. 51, p. 651 (March 28, 1908). (Brief data on the elements of cost of plant.)

ELECTRIC LIGHT AND POWER—SPECIAL CASES—(*Continued*).

New York City.

ACCOUNTING FOR DEPRECIATION; by H. M. Edwards. *Electric Railway Journal*, v. 37, p. 972 (June 3, 1911). (Method used by the New York Edison Co.)

REGULATED ELECTRIC LIGHT ACCOUNTING; by H. M. Edwards. National Electric Light Association, Thirty-fifth Convention, 1912, Papers, Reports and Discussions, v. 4, p. 106. (On the uniform system of accounts for electrical corporations as prescribed by the Public Service Commission, State of New York, First District, and the petition to modify it.)

——Abstract. *Electric Railway Journal*, v. 39, p. 1029 (June 15, 1912).

New York State.

NEW YORK PUBLIC SERVICE COMMISSION TENTATIVE ACCOUNTS FOR ELECTRICAL and Gas Corporations. *Electric Railway Review*, v. 19, p. 532 (May 2, 1908). (Classification for accounts prepared by W. J. Meyers; abstract of some features of the system.)

PETITION FOR CHANGES IN TREATMENT OF DEPRECIATION IN NEW YORK. *Electrical World*, v. 58, p. 1420 (Dec. 9, 1911). (Petition filed by various lighting companies with New York Public Service Commission, Second District; one page.)

STANDARD ACCOUNTING CONFERENCE. *Progressive Age*, v. 26, p. 267 (May 1, 1908). (On report of classification of accounts, gas and electric companies, by the Public Service Commission, State of New York, Second District.)

***STATE OF NEW YORK, SECOND ANNUAL REPORT OF THE PUBLIC SERVICE** Commission, Second District, for the Year Ending Dec. 31, 1908; v. 2, Uniform System of Accounts. Albany, 1909. (Classification of accounts for street railroads, gas and electrical corporations; general amortization account, including amount estimated for wear, tear and obsolescence of plant.)

Pacific Gas & Electric Co.

PACIFIC GAS RATE VALUATION; by John A. Britton. *Progressive Age*, v. 30, p. 330 (April 15, 1912). (Includes cost of electric energy, depreciation and administration.)

Pasadena, Cal.

PASADENA MUNICIPAL LIGHTING PLANT. *Municipal Engineering*, v. 44, p. 505 (June, 1913). (Capitalization, depreciation allowance, etc., in relation to rate regulation.)

Red Cedar Valley, Wis.

***IN RE APPLICATION OF THE RED CEDAR VALLEY ELECTRIC COMPANY FOR** Authority to Increase its Rates; Decided June 14, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 6, p. 717. Madison, Wis., 1912. (Contains Company's statement of the value of the physical property of the plant.)

Ripon, Wis.

***CITY OF RIPON VS. RIPON LIGHT AND WATER COMPANY**; Decided March 28, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 1. Madison, Wis., 1911. (Data on vaulation of the water, light and electric plants.)

St. Louis, Mo.

REPORT OF ST. LOUIS PUBLIC SERVICE COMMISSION TO THE MUNICIPAL Assembly of St. Louis on Rates for Electric Light and Power. St. Louis, 1911. (Contains description of methods of appraisal of the property of the Union Electric Light & Power Co.)

San Francisco, Cal.

UNIT GENERATING AND DISTRIBUTION COSTS OF THE PACIFIC GAS & ELECtric Company in San Francisco. *Electrical World*, v. 59, p. 790 (April 13, 1912).

Sheboygan, Wis.

***CITY OF SHEBOYGAN VS. SHEBOYGAN RAILWAY AND ELECTRIC COMPANY**; Submitted Oct. 18, 1910, Decided Feb. 3, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 6, p. 353. Madison, Wis., 1912. (Company's estimate of investment and annual expenses chargeable to street lighting.)

Superior Water, Light & Power Co.

***ESTIMATING THE RATE OF "REASONABLE RETURN" FOR A PUBLIC UTILITY.** *Engineering and Contracting*, v. 39, p. 482 (April 30, 1913). (Argument submitted to the Wisconsin Railroad Commission, for the Superior Water, Light & Power Co., giving analysis of the rate of fair return for capital invested.)

ELECTRIC LIGHT AND POWER—SPECIAL CASES—(*Continued*).

***PUBLIC SERVICE COMMISSION NEWS.** *Electrical World,* v. 60, p. 1136 (Nov. 30, 1912). (Basis of valuation of electric plant in investigation of the revenues of the Superior Water, Light & Power Co.)

Wakefield, Mass.

DEPRECIATION OF MUNICIPAL LIGHTING PLANTS. (Editorial.) *Electrical Review and Western Electrician,* v. 53, p. 493 (Oct 3, 1908). (Inadequacy of allowance for depreciation in plant at Wakefield, Mass.)

Waupaca, Wis.

***IN RE JOINT APPLICATION OF THE WAUPACA ELECTRIC LIGHT AND RAILway Company and the City of Waupaca to the Effect that the Railroad Commission Act as Arbitrator in Certain Matters Pertaining to Street Lighting in the City of Waupaca; Submitted Dec. 15, 1910, Decided Feb. 21, 1912.** In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 8, p. 586. Madison, Wis., 1912. (Total reproduction cost, present value and cost of operation were ascertained and apportioned between street lighting and all other service.)

West Ham, England.

IS DEPRECIATION AS SUCH, NEEDED? (Editorial.) *Municipal Journal* (London), v. 12, p. 699 (July 31, 1903). (Relates to West Ham Corporation electric lighting plant.)

Wisconsin.

***ADJUSTMENT OF ELECTRIC LIGHTING RATE.** *Power,* v. 35, p. 498 (April 9, 1912). (Extracts from the reports of the Wisconsin Railway Commission regarding the influence of various fixed charges upon the rates.)

METHODS OF OBTAINING COST OF ELECTRIC LIGHTING SERVICE TO CONsumers Based on Decisions of the Wisconsin Railroad Commission. *Engineering and Contracting,* v. 37, p. 48 (Jan. 10, 1912). (Four pages.)

***UNIFORM CLASSIFICATION OF ACCOUNTS FOR ELECTRIC UTILITIES PREscribed by the Railroad Commission of Wisconsin, Dec., 1908. Edition 3.** Madison, Wis., 1912. (Treats of tangible and intangible property, reserve accounts, etc.)

WISCONSIN CLASSIFICATION OF ELECTRIC ACCOUNTS. *Electrical World,* v. 53, p. 503 (Feb. 25, 1909). (Classification prepared by the Wisconsin Railroad Commission.)

Worcester, Mass.

THE APPRAISAL OF STREET LIGHTING SERVICE. *Engineering Record,* v. 66, p. 104 (July 27, 1912). (Decision by the Massachusetts Gas and Electric Light Commission in the Worcester street lighting case.)

York, England.

DEPRECIATION IN ELECTRIC LIGHTING. (Editorial.) *Municipal Journal* (London), v. 12, p. 12 (Jan. 2, 1903). (Policy of York Corporation; very brief.)

GAS-WORKS—GENERAL.

DEPRECIATION; by John A. Britton. *Progressive Age,* v. 26, p. 379 (June 15, 1908). (Depreciation as an item to be considered in fixing gas rates.)

DEPRECIATION ALLOWANCE FOR INCOME-TAX. (Editorial.) *Journal of Gas Lighting,* v. 108, p. 517 (Nov. 23, 1909). (Protest against action of Inland Revenue authorities on depreciation in gas plants.)

DEPRECIATION: AN ITEM IN THE MANUFACTURE AND SALE OF GAS; by Royal Schacklette. *American Gas Light Journal,* v. 93, p. 106 (July 8, 1910). (Considers depreciation in the various parts of the plant.)

DEPRECIATION AND INCOME-TAX. *Journal of Gas Lighting,* v. 118, p. 886 (June 18, 1912). (Refers to depreciation in gas plants.)

DEPRECIATION AND INCOME-TAX, THE CASE OF MUNICIPAL UNDERTAKINGS. *Journal of Gas Lighting,* v. 121, p. 48 (Jan. 7, 1913). (Methods of allowing for depreciation in gas, water and electric lighting undertakings.)

DEPRECIATION AND KINDRED MATTERS; by Charles H. Armstrong. *Journal of Gas Lighting,* v. 102, p. 223 (April 28, 1908). (A paper read at a meeting of the North of England Gas Managers' Association.)

DEPRECIATION IN GAS PLANTS; by John I. Beggs. *American Gas Light Journal,* v. 89, p. 5 (July 6, 1908). (Address before the Wisconsin Gas Association.)

——Maintenance of Plant; by John I. Beggs. *Progressive Age,* v. 26, p. 427 (July 15, 1908).

GAS-WORKS—GENERAL—(*Continued*).

DEPRECIATION IN GAS PLANTS; by R. W. Prosser. *American Gas Light Journal*, v. 76, p. 767 (May 26, 1902). (Gives tables of life of apparatus of a gas works and annual charge for depreciation; one and one-half pages.)

DEPRECIATION IN GAS WORKS. *Progressive Age*, v. 30, p. 870 (Oct. 15, 1912).

INAUGURAL ADDRESS OF THE PRESIDENT; by Alexander Wilson. *Transactions*, Institution of Gas Engineers, 1911, p. 51. (Takes up the question of depreciation; four pages.)

——Abstract. *Journal of Gas Lighting*, v. 114, p. 728 (June 13, 1911).

THE PRESENT AND GROWING NEED FOR THE GENERAL ADOPTION OF A Uniform System of Records and Accounts; by A. C. Humphreys. *Proceedings*, New England Association of Gas Engineers, 1907, p. 146.

RATES AND RATE MAKING; by John F. Druar. *Journal*, Association of Engineering Societies, v. 50, p. 221 (May, 1913). (Discusses the valuation of a combined electrical and gas property for determining the legitimate capital, upon which capital a certain return should be received.)

REASONABLE GAS RATES AND THEIR DETERMINATION; by C. L. Cory. *Progressive Age*, v. 30, p. 964 (Nov. 15, 1912). (Discusses rates of return and value of gas plant.)

REVIEW OF RECENT DECISIONS OF PUBLIC SERVICE COMMISSIONS WHICH Affect Gas Companies; by A. E. Forstall. *Proceedings*, American Gas Institute, v. 5, p. 460 (1910). (Discusses methods of valuation considering physical value, cost of reproduction, depreciation, etc.)

——*American Gas Light Journal*, v. 94, p. 148 (Jan. 3, 1911).

GAS-WORKS—SPECIAL CASES.

Baltimore, Md.

STOCK AND FAIR VALUE. *Public Service Regulation*, v. 1, p. 550 (Aug., 1912). (Baltimore gas and electric rate hearing, before the Maryland Commission.)

Beloit, Wis.

BELOIT UTILITY DECISION BY RAILROAD COMMISSION OF WISCONSIN. *American Gas Light Journal*, v. 95, p. 162 (Sept. 11, 1911). (Gives estimate in valuation of the property of the Beloit Water, Gas & Electric Co.)

***CITY OF BELOIT VS. BELOIT WATER, GAS AND ELECTRIC COMPANY**; Decided July 19, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 220. Madison, Wis., 1912. (Value of gas plant is given.)

FIXING NORMAL OPERATING COSTS; by Frank A. Newton. *Engineering Record*, v. 65, p. 258 (March 9, 1912). (Comments on the decision in the case of the City of Beloit *vs.* the Beloit Water, Gas & Electric Co.)

Brooklyn, N. Y.

PAVING OVER GAS MAINS AND CAPITALIZATION OF PROFITABLE CONTRACTS as Elements in Rate Making: Decision of the New York Public Service Commission in the Kings County Lighting Case. *Engineering News*, v. 66, p. 604 (Nov. 16, 1911). (Discusses paving over mains and capitalization of contracts as factors in the valuation of the property, citing Supreme Court decisions.)

***RE MAYHEW V. KINGS CO. LIGHTING CO. (CASE 1273).** Reports of Decisions of the Public Service Commission for the First District, of the State of New York, v. 2, p. 659. New York, 1912. (Details of valuation of property of the Kings County Lighting Co. and methods of estimating it.)

California.

PACIFIC GAS RATE VALUATION; by John A. Britton. *Progressive Age*, v. 30, p. 330 (April 15, 1912). (Cost of gas in holder, 1911, cost of gas distribution, cost of administration and depreciation, and cost of electric energy, depreciation and administration, Pacific Gas & Electric Co.)

***UNIFORM CLASSIFICATION OF ACCOUNTS FOR GAS CORPORATIONS PRE**scribed by the Railroad Commission of the State of California; Adopted Oct. 23, 1912, Effective, Jan. 1, 1913. Sacramento, 1912. (Classification of intangible and tangible capital, including franchise, patent rights, physical property, etc.)

Cedar Rapids, Iowa.

FAIR RATES FOR PUBLIC SERVICE. (Letter.) *Engineering Record*, v. 59, p. 699 (May 29, 1909). (Decision of Iowa Supreme Court on valuation of the Cedar Rapids Gas Light Co.)

GAS-WORKS—SPECIAL CASES—(*Continued*).

Chicago, Ill.

APPRAISAL OF GAS PROPERTIES IN CHICAGO, AND INVESTIGATION AS TO Reasonable Rates for Gas: Valuation of Physical Property; by William J. Hagenah. *Engineering and Contracting,* v. 35, p. 572 (May 17, 1911). (Twelve pages.)

——Editorial. The Chicago Gas Appraisal. *Engineering and Contracting,* v. 35, p. 557 (May 17, 1911). (One column.)

INVESTIGATION OF THE PEOPLES GAS LIGHT AND COKE COMPANY FOR THE Council Committee on Gas, Oil and Electric Light, Chicago; by William J. Hagenah. Journal of the Proceedings of the Council of Chicago, July 10, 1911, p. 840. (Gives full report on the valuation of the Company; 89 pages.)

PRICE OF GAS IN CHICAGO. Journal of the Proceedings of the City Council of Chicago, July 10, 1911, p. 820. (Report of Edward W. Bemis commenting on report by W. J. Hagenah.)

——Editorial. Gas Rates in Chicago. *Municipal Engineering,* v. 41, p. 206 (Sept., 1911).

Chippewa Falls, Wis.

*T. J. CUNNINGHAM ET AL. VS. CHIPPEWA FALLS WATER WORKS AND LIGHTing Company: In Re Investigation by the Railroad Commission of Wisconsin of Rates Charged by the Chippewa Falls Water Works and Lighting Company; In Re Valuation of the Property of the Chippewa Falls Water Works and Lighting Company; Decided June 14, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 302. Madison, Wis., 1911. (Contains data on the value of the Gas Department.)

District of Columbia.

*UNIFORM SYSTEM OF ACCOUNTS FOR GAS CORPORATIONS AND ELECTRIC Corporations in the District of Columbia as Prescribed by the Interstate Commerce Commission, pp. 29, 47, 55, 67. Washington, 1909. (Provision for amortization of plant, which includes monthly charges of the amount estimated to be necessary to cover wear, tear and obsolescence.)

Galesburg, Ill.

THE GAS FIGHT AT GALESBURG. *Public Service,* v. 4, p. 67 (March, 1908). (Report by Byllesby & Co. on the valuation of the gas plant at Galesburg, Ill.)

La Crosse, Wis.

DEPRECIATION AS A FACTOR IN RATES. (Editorial.) *Electric Railway Review,* v. 18, p. 752 (Nov. 9, 1907). (Petition of La Crosse Gas & Electric Co.)

*IN RE APPLICATION OF THE LA CROSSE GAS AND ELECTRIC COMPANY FOR Authority to Increase Its Rates; Decided Nov. 17, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 8, pp. 138, 156, 170, 179, 195, 224. Madison, Wis., 1912. (Physical value of plant, original cost and effect of allowance for going value.)

*THE MEANING OF "ACTUAL STATION OPERATING COSTS." *Engineering Record,* v. 65, p. 191 (Feb. 17, 1912). (Decision of the Railroad Commission of Wisconsin in the case of the La Crosse Gas & Electric Co.)

Madison, Wis.

THE APPRAISAL OF THE MADISON GAS AND ELECTRIC CO. *Engineering and Contracting,* v. 35, p. 747 (June 28, 1911). (Appraisal made by the Wisconsin State Railroad Commission; three pages.)

GOING VALUE OF PUBLIC SERVICE CORPORATIONS, AS DETERMINED BY THE Wisconsin Railroad Commission in the Madison Gas and Electric Case. *Engineering and Contracting,* v. 35, p. 691 (June 14, 1911). (On the decision rendered in the case of the State Journal Printing Co. *et al. vs.* the Madison Gas & Electric Co.; four pages.)

*IMPORTANT DECISION BY WISCONSIN COMMISSION ON CENTRAL-STATION Valuation and Rate-Making. *Electrical World,* v. 55, p. 675 (March 17, 1910). (Decision by the Wisconsin Railroad Commission on "going value", rate of return, etc., in the case of State Journal Printing Co. *et al. vs.* Madison Gas & Electric Co.)

*IN RE INVESTIGATION, ON MOTION OF THE COMMISSION, OF THE RATES, Rules and Regulations of the Madison Gas and Electric Company; Decided July 5, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 152. Madison, Wis., 1912. (Gives cost of reproduction new and existing value of the Madison gas and electric lighting plant.)

REASONABLE RATES FOR GAS AND ELECTRICITY AS DETERMINED BY THE Wisconsin Railroad Commission for the City of Madison. *Engineering News,* v. 63, p. 380 (March 31, 1910). (State Journal Printing Co. *vs.* Madison Gas & Electric Co.; gives the method of valuation and related investigations.)

GAS-WORKS—SPECIAL CASES—(*Continued*).

***STATE JOURNAL PRINTING COMPANY ET AL. VS. MADISON GAS AND ELEC**tric Company; Decided March 8, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 4, p. 501. Madison, Wis., 1910. (Method of arriving at valuation of gas and electric plants; also method of valuing real estate.)

Manitowoc, Wis.

***IN RE APPLICATION OF THE MANITOWOC GAS COMPANY FOR AUTHORITY** to Equalize Rates; Submitted Sept. 28, 1908, Decided Dec. 17, 1908. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 3, p. 163. Madison, Wis., 1910. (Data on the physical valuation of the property.)

***PHILOSOPHY OF GAS RATES, DECISION OF WISCONSIN RAILROAD COMMIS**sion Authorizing Manitowoc Gas Company to Increase Its Rates. *Public Service*, v. 6, p. 58 (Feb., 1909). (Contains brief reference to depreciation.)

Neenah, Wis.

***CITY OF NEENAH VS. THE WISCONSIN TRACTION, LIGHT, HEAT AND POWER** Company; Decided Aug. 4, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 479. Madison, Wis., 1912. (Contains a tentative valuation of the physical property of the Gas Department of the Wisconsin Traction, Light, Heat & Power Co.)

New Jersey.

***GAS RATE DECISIONS IN NEW JERSEY.** *Electric Railway Journal*, v. 41, p. 35 (Jan. 4, 1913). (Decision of New Jersey Public Utilities Commission; relates to valuation of property of the Public Service Gas Co. in the Passaic District.)

***GOING CONCERN VALUE.** (Editorial Note.) *American Gas Light Journal*, v. 98, p. 208 (March 31, 1913). (Brief extract from decision by the Board of Public Utility Commissioners of New Jersey.)

New York City.

***DECISION OF THE SUPREME COURT OF THE UNITED STATES, THE PUBLIC** Service Commission *vs.* the Consolidated Gas Company of New York. National Electric Light Association, Thirty-second Convention, 1909, Papers, Reports and Discussions, v. 3, pp. 319, 324. (Considers valuation of franchises and value of the Company's property.)

THE EIGHTY-CENT GAS DECISION OF THE SUPREME COURT. (Editorial.) *Engineering Record*, v. 59, p. 31 (Jan. 9, 1909).

FULL TEXT OF THE OPINION OF THE SUPREME COURT OF THE UNITED STATES in the Matter of the New York Legislature's Order Fixing the Gas Rates in New York City at 80 Cents per 1 000 Cubic Feet. *American Gas Light Journal*, v. 90, p. 99 (Jan. 18, 1909). (Considers value of property and franchise; three pages.)

New York State.

***NEW YORK PUBLIC SERVICE COMMISSION TENTATIVE ACCOUNTS FOR ELEC**trical and Gas Corporations. *Electric Railway Review*, v. 19, p. 532 (May 2, 1908). (Classification prepared by W. J. Meyers; abstract of some features of the system.)

***STANDARD ACCOUNTING CONFERENCE.** *Progressive Age*, v. 26, p. 267 (May 1, 1908). (On report of classification of accounts, gas and electric companies, by Public Service Commission, State of New York, Second District.)

***STATE OF NEW YORK, SECOND ANNUAL REPORT OF THE PUBLIC SERVICE** Commission, Second District, for the Year Ending Dec. 31, 1908; v. 2, Uniform System of Accounts. Albany, 1909. (Classification of accounts for street railroads, gas and electrical corporations; general amortization account, including amount estimated for wear, tear and obsolescence of plant.)

Passaic, N. J.

PUBLIC SERVICE GAS COMPANY RATE CASE; by Frank Bergen. *Aera*, v. 1, p. 465 (Jan., 1913). (Valuation of the property of the Passaic Division of the Public Service Gas Co., and discussion of methods.)

Peoria, Ill.

IN THE CIRCUIT COURT OF THE UNITED STATES, NORTHERN DISTRICT OF Illinois, Southern Division, Peoria Gas and Electric Company, Complainant, *vs.* City of Peoria, Defendant, in Chancery for Injunction, Etc.: Special Master's Report. Barnard & Miller, Chicago, 1903. (Contains data on methods of valuation of gas plants.)

Queens Borough Gas & Electric Co.

***RATES OF THE QUEENS BOROUGH GAS AND ELECTRIC CO.: THE FIRST COM**modity-Rate Case of the Public Service Commission for the First District of New York. *Engineering News*, v. 66, p. 266 (Aug. 31, 1911). (Method of appraisal of property; land appraisal, development costs, working capital, going value, unimpaired investment and annual depreciation.)

GAS-WORKS—SPECIAL CASES—(*Continued*).

***RE RATES OF QUEENS BOROUGH GAS AND ELECTRIC COMPANY (CASES 1224** and 1225). Reports of Decisions of the Public Service Commission of the First District of the State of New York, v. 2, p. 544. New York, 1912. (Methods of determining value of gas property.)

Racine, Wis.

***CITY OF RACINE VS. RACINE GAS LIGHT COMPANY;** Decided Jan. 27, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 6, p. 229. Madison, Wis., 1912. (A tentative valuation of the physical property of the Racine Gas Light Co., including land, breakwater, mains and paving, and the going value of the plant.)

DECISION OF THE RAILROAD COMMISSION OF WISCONSIN IN THE RACINE Case. *American Gas Light Journal,* v. 94, p. 547 (March 20, 1911). (Gives valuation of the Racine Gas Light Co.'s plant.)

Saginaw, Mich.

SAGINAW GAS DECISION. *Progressive Age,* v. 26, p. 722 (Dec. 15, 1908). (Contains data on the valuation of the gas plant.)

St. Paul, Minn.

THE FAIR PRICE OF GAS AT ST. PAUL; by R. S. Feurtado. *Public Service,* v. 3, p. 173 (Dec., 1907). (Valuation of the plant of the St. Paul Gas Light Co.)

San Francisco, Cal.

THE COST OF GAS IN SAN FRANCISCO, REPORT TO THE BOARD OF SUPER-visors, May 18, 1908; by C. D. Stuart. *Public Service,* v. 5, p. 20 (July, 1908). (On the valuation of the gas plant.)

Sheboygan, Wis.

THE SHEBOYGAN, WIS., GAS CASE. *Public Service Regulation,* v. 1, p. 695 (Oct., 1912). (Opinion in case of Sheboygan Gas Light Co., *re* valuation of the property.)

Superior, Wis.

***ESTIMATING THE RATE OF "REASONABLE RETURN" FOR A PUBLIC UTILITY.** *Engineering and Contracting,* v. 39, p. 482 (April 30, 1913). (Analysis of the rate of return; argument submitted to the Wisconsin Railroad Commission by the Superior Water, Light & Power Co.)

***WISCONSIN RATES.** *Progressive Age,* v. 30, p. 1075 (Dec. 16, 1912). (Decision of the Railroad Commission of Wisconsin in the application of the Superior Commercial Club *et al. vs.* the Superior Water, Light & Power Co.)

GAS-WORKS—UNVERIFIED REFERENCES.

DEPRECIATION USED FOR UNIFORM ACCOUNTS. *Public Service,* v. 2, p. 147 (May, 1907). (On depreciation in gas plants.)

DES MOINES GAS CO. VS. CITY OF DES MOINES ET AL.; In the District Court of the United States in and for the Southern District of Iowa, Central Division: Complainant's Brief on Exception to Master's Report.

PRIVILEGE BECOMES PROPERTY UNDER THE FOURTEENTH AMENDMENT: The Consolidated Gas Decision; by Jesse F. Orton. *Independent,* p. 719 (Oct. 12, 1911).

MINING.

AMERICAN IRON-ORE RESERVES; by Edwin C. Eckel. *Engineering Magazine,* v. 44, p. 7 (Oct., 1912). (Bases for valuation of iron-ore properties.)

***APPRAISAL OF MINING PROPERTIES OF MICHIGAN BY THE STATE BOARD** of Tax Commissioners. Lansing, Mich., 1911, James H. Thompson, Chairman, State Board of Tax Commissioners. (Gives inventory and appraisal of all the mining properties of the State; eighty-two pages.)

DEPRECIATION AS AFFECTING COLLIERIES AND IRON WORKS; by H. A. Foster. *Iron and Coal Trades Review,* v. 65, p. 1303 (Nov. 21, 1902). (Some of the methods of treating the subject of depreciation.)

STOCK VALUE AND MINE VALUE; by Alfred C. Lane. *Canadian Mining Journal,* v. 32, pp. 691, 729, 775 (Nov. 1, 15, Dec. 1, 1911). (Elements to be considered in the valuation of mines; return, stability, marketability, activity and control.)

THE VALUATION OF MINES; by T. A. Rickard. *Mining and Scientific Press,* v. 106, p. 766 (May 24, 1913).

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ACCOUNTING DEPRECIATION. *Railroad Age Gazette,* v. 45, p. 415 (July 3, 1908). (On the relation of valuation to depreciation; one column.)

AMERICAN TRANSPORTATION QUESTION, p. 81; by Samuel O. Dunn. D. Appleton & Co., New York, 1912. (On the theory of railroad valuation; forty-two pages.)

AN AMERICAN TRANSPORTATION SYSTEM, p. 316; by George A. Rankin. G. P. Putnam's Sons, New York, 1909. (Contains a discussion of the appraisement of railroads; seventeen pages.)

APPORTIONMENT BETWEEN STATE AND INTERSTATE TRAFFIC OF RAILway Property Devoted to the Public Service; by Thomas D. O'Brien. *Proceedings,* Annual Convention of the National Association of Railway Commissioners, 1909, p. 306. (Method of valuation by reproduction cost; three pages.)

APPRAISED VALUE OF THE RAILWAYS IN FIVE STATES AND THE PROBABLE Cost of Reproducing All Railways in America. *Engineering-Contracting,* v. 34, p. 89 (Aug. 3, 1910). (Results and comparisons of State valuations.)

THE ARBITRARY DEPRECIATION CHARGE; by F. A. Delano. *Railroad Gazette,* v. 44, p. 681 (May 15, 1908). (A paragraph from the *Wall Street Journal.*)

ASSIGNMENT OF STEAM AND ELECTRIC LOCOMOTIVES, PASSENGER AND Freight Train Cars and Work Equipment Cost to the Several States and to Operating Divisions Within States. *Engineering and Contracting,* v. 39, p. 724 (June 25, 1913). (Abstract of paper by A. I. T. Thompson read before the Mississippi Valley States Conference.)

THE ASSIGNMENT OF VALUATION OF FACILITIES TO MORE THAN ONE STATE. *Engineering and Contracting,* v. 39, p. 726 (June 25, 1913). (A plan for apportioning value of general railroad shops between States; abstract of paper by Hugh H. Bryant read before the Mississippi Valley States Conference.)

BASIS OF VALUATION AS BETWEEN INTRASTATE AND INTERSTATE BUSINESS. *Railway Age Gazette,* v. 46, p. 319 (Feb. 12, 1909). (One paragraph.)

COMMERCIAL VALUATION OF RAILWAY OPERATING PROPERTY IN THE United States, 1904; by Henry C. Adams. U. S. Bureau of the Census, Bulletin No. 21, Washington, 1905. (Reports by Prof. Henry C. Adams, Prof. B. H. Meyer, William J. Meyers and others; eighty-eight pages.)

——Abstract. Railroads Valuations in State Reports. *Railroad Gazette,* v. 39, p. 226 (Sept. 8, 1905).

——Editorial. The Census Office Railroad Valuation. *Railroad Gazette,* v. 39, p. 194 (Sept. 1, 1905).

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COST, CAPITALIZATION AND ESTIMATED VALUE OF AMERICAN RAILWAYS: An Analysis of Current Fallacies; by Slason Thompson. Edition 3. Bureau of Railway News, Chicago, 1908. (Aims to show that the value of railway properties in the United States exceeds their total net capitalization.)

——Editorial. Cost, Capitalization and Values of American Railways. *Railway Age,* v. 44, p. 710 (Nov. 22, 1907).

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DEDUCTIONS FOR OBSOLESCENSE JUSTIFIED. *Railway Age Gazette,* v. 49, p. 1093 (Dec. 2, 1910). (Decision of the Supreme Court of New York regarding the value of franchises and allowance for depreciation in taxation.)

DEPRECIATION; by S. M. Hudson. *Railway Age,* v. 44, p. 175 (Aug. 9, 1907). (General discussion of depreciation; one and one-half pages.)

DEPRECIATION IN RAILWAY ACCOUNTING. *Railway Age,* v. 43, p. 728 (May 10, 1907). (Recommendations of Interstate Commerce Commission; one page.)

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DEPRECIATION IN STEAM RAILWAY ACCOUNTING. *Electric Railway Journal,* v. 32, p. 748 (Oct. 3, 1908). (Memorandum compiled by Special Committee on Relations with Interstate Commerce Commission of the American Railway Association; one page.)

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DETERMINING A REASONABLE RATE. (Editorial.) *Railway World,* v. 56, p. 344 (April 19, 1912). (Comments on opinion of Judge Thomas G. Jones, of Alabama, on the cost of reproduction of a railroad, the basis for reasonable rates.)

DEVELOPMENT OF THE FREIGHT RATE HEARING REGARDING THE PHYSICAL Valuation of Railways. (Editorial.) *Engineering-Contracting,* v. 34, p. 263 (Sept. 28, 1910). (One and one-half columns.)

DISCUSSION OF REPORT OF COMMITTEE ON RATES AND RATE MAKING; by M. R. Maltbie. *Proceedings,* Annual Convention of the National Association of Railway Commissioners, 1910, pp. 200, 204. (On "market value," depreciation, and methods of making valuation; five pages.)

THE ECONOMICS OF RAILROAD CONSTRUCTION, p. 41; by Walter Loring Webb. John Wiley & Sons, New York, 1906. (Contains a chapter on the valuation of railway property.)

EQUIPMENT ACCOUNT FOR EACH CAR AND LOCOMOTIVE. *Railway Age Gazette,* v. 43, p. 640 (Nov. 29, 1907). (Method of keeping a depreciation account.)

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FAIR RETURN ON THE VALUE OF PROPERTY: A FALLACIOUS STANDARD. *Railway Age Gazette,* v. 48, p. 1129 (May 6, 1910). (From an address by Walker D. Hines before the Traffic Club of Pittsburgh.)

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GOVERNMENT SUPERVISION OF RAILWAY ACCOUNTS; by Henry C. Adams. *Electric Railway Review,* v. 19, p. 43 (Jan. 11, 1908). (Abstract of paper read before the Association of American Government Accountants.)

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HENRY C. ADAMS ON RAILWAY VALUATION. *Railway World,* v. 51, p. 467 (June 7, 1907). (A brief analysis of the elements of valuation.)

HENRY FINK ON DANGER OF RAILWAY VALUATION. *Railway Age,* v. 43, p. 680 (April 26, 1907). (Declares there is no relation between valuation and rate regulation.)

INCOME ACCOUNT OF RAILWAYS. (Letter); by Frank May. *Railway Age,* v. 45, p. 560 (April 17, 1908). (How depreciation charges should be made.)

INITIAL COST, COST OF MAINTENANCE AND DEPRECIATION OF WOODEN Passenger Cars. *Engineering-Contracting,* v. 34, p. 215 (Sept. 7, 1910). (Rate of depreciation per year of different classes of cars.)

LET THE GOVERNMENT GO AHEAD AND PROSECUTE AND APPRAISE THE Railways. (Editorial.) *Railway Age Gazette,* v. 49, p. 566 (Sept. 30, 1910). (Relation between valuation and rates.)

MAINTENANCE CHARGES AND DIVIDENDS. (Editorial.) *Engineering Record,* v. 55, p. 86, (Jan. 26, 1907). (Alludes to comparison of repair costs in relation to traffic; one column.)

MAY RESERVE FUND TO RENEW ABSOLUTE EQUIPMENT. (Editorial.) *Electric Traction Weekly,* v. 6, p. 1473 (Dec. 3, 1910). (Decision by Supreme Court; article states that this is a new principle on valuation of special franchises.)

A METHOD OF APPRAISING NON-PHYSICAL RAILWAY VALUES. (Editorial.) *Engineering-Contracting,* v. 34, p. 517 (Dec. 14, 1910). (Comments on paper by Henry Earle Riggs on valuation of public utilities and on the method employed by Prof. Henry C. Adams in valuation of railroads.)

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——Letter. State Valuation of Railroads; by Carl Tombo. *Railway Age,* v. 44, p. 348 (Sept. 13, 1907).

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NATIONAL VALUATION CONVENTION URGED, CONCERTED ACTION SHOULD BE Taken to Make Appraisal of Railways Economical, Intelligent and Just; by H. Bortin. *Railway Age Gazette,* v. 54, p. 836 (April 11, 1913).

THE NECESSITY OF DEPRECIATION RESERVES; by Henry L. Gray. *Railway Age Gazette,* v. 48, p. 1297 (May 27, 1910). (Two pages.)

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——Editorial. Depreciation Reserves. *Railway Age Gazette*, v. 48, p. 1290 (May 27, 1910).

——Letters. Depreciation Reserves. *Railway Age Gazette*, v. 49, p. 66 (July 8, 1910).

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——Abstract. Valuation of Steam Railroads. *Railway Age Gazette*, v. 67, pp. 654, 692 (June 14, 21, 1913).

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THE PROBLEM OF RAILWAY VALUATION; by Logan G. McPherson. *Railway Age Gazette*, v. 54, p. 1131 (May 23, 1913). (The change in the attitude of the public toward the carriers, and the various difficult questions it has raised.)

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PROPOSED VALUATION OF RAILROAD PROPERTY. (Editorial.) *Railway Age Gazette*, v. 42, p. 293 (March 8, 1907). (Concerning valuation as a basis for rate-making; one and one-half columns.)

RAILROAD ACCOUNTING AND THE HEPBURN LAW; by Arthur C. Graves. *Railroad Age Gazette*, v. 45, pp. 1543, 1597; v. 46, p. 18 (Dec. 11, 18, 1908; Jan. 1, 1909). (A protest against the requirements of the Government in railroad accounting.)

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RAILWAY ATTITUDE TOWARD VALUATION OF RAILWAYS. *Railway Age Gazette*, v. 49, p. 1137 (Dec. 16, 1910). (Arguments against a mere physical valuation; one page.)

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——*Railway Age Gazette*, v. 48, p. 944 (April 8, 1910).

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——Abstract. *Electric Railway Journal*, v. 34, p. 1146 (Dec. 4, 1909).

RAILWAY VALUATION AGAIN. *Railway and Engineering Review*, v. 50, p. 1172 (Dec. 24, 1910). (Comments on statement of Judson C. Clements before the Railways Securities Commission; from *New York Sun*, Dec. 3, 1910.)

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——Abstract of report for 1905. *Railway Age*, v. 40, p. 289 (Sept. 8, 1905).

——Abstract of report for 1910. *Electric Railway Journal*, v. 36, pp. 1062, 1192 (Nov. 26, Dec. 17, 1910); *Railway and Engineering Review*, v. 50, p. 1130 (Dec. 10, 1910).

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——Abstract of report for 1912. *Electric Railway Journal*, v. 40, p. 1065 (Nov. 23, 1912). (Treats of allowance for contingencies, interest during construction, basis for valuation, cost of reproduction new and original investment and fair value.)

——Editorials on report for 1912. Railway Valuation. *Electric Railway Journal*, v. 40, p. 1051 (Nov. 23, 1912); Valuation by the National and the State Governments. *Electric Railway Journal*, v. 40, p. 1139 (Dec. 7, 1912).

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——Abstract. *Electric Railway Journal*, v. 38, p. 812 (Oct. 13, 1911).

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——Criticism. Mr. Loweth on Depreciation in Valuation. (Letter); by C. F. Loweth. *Railway Age Gazette*, v. 54, p. 1536 (June 20, 1913). (Criticism on editorial in issue of May 30th, 1913.)

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SOME NEGLECTED FACTORS OF FAIR VALUATION. (Editorial.) *Railway Age Gazette*, v. 46, p. 441 (March 5, 1909). (Concerning the relation between physical valuations as made by State Commissions and rate regulation.)

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STATISTICS AS TO THE LIFE OF STEEL RAILWAY BRIDGES. *Engineering-Contracting*, v. 30, p. 227 (Oct. 7, 1908). (Reference to depreciation of bridges and table showing life of ten railway bridges.)

AN UNSEEN FACTOR IN RAILWAY VALUATION. (Editorial.) *Railway Age Gazette*, v. 50, p. 821 (April 7, 1911). (In regard to the element which represents investment extinct and destroyed in the various railway re-organizations.)

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THE VALUATION OF AMERICAN RAILWAYS. (Editorial.) *Railway World*, v. 51, p. 410 (May 17, 1907). (Discusses value regarded as original or reproduction cost and as capitalization on the net earnings of the road.)

VALUATION OF PUBLIC SERVICE CORPORATIONS; by W. H. Williams. American Economic Association, New York, 1909. (Discusses the valuation of railroad taxes, rates, capitalization, etc.; fifty-one pages.)

——Abstract. *Electric Railway Journal*, v. 35, p. 76 (Jan. 8, 1910).

VALUATION OF RAILROAD PROPERTY; by Henry Fink. *Railway Age Gazette*, v. 45, pp. 587, 627 (July 24, 31, 1908). (Discusses physical valuation, capitalization and rate-making.)

VALUATION OF RAILROAD PROPERTY FOR LOCAL TAXATION. (Letter.) *Railroad Gazette*, v. 29, p. 863 (Dec. 10, 1897). (Decisions on valuation of railroads, based on opposing principles, cost of reproduction and earning capacity.)

VALUATION OF RAILROADS. (Editorial.) *Railway Age Gazette*, v. 42, p. 730 (May 31, 1910). (History of attempts to fix value of railroad properties, and statement that this cannot be done on a physical basis; one page.)

VALUATION OF RAILWAY PROPERTIES; by Robert Yates. *Railway Age Gazette*, v. 47, p. 975 (Nov. 19, 1909). (Considers a few of the principal elements of construction showing the cost value and depreciated value.)

VALUATION OF RAILWAYS. *Railway Age Gazette*, v. 46, pp. 173, 219, 261, 312 (Jan. 22, 29, Feb. 5, 12, 1909). (Full discussion of valuation in relation to State Railway Commissions, rates, etc.)

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——Abstract. Valuation of the Railways of the United States. *Railway Age*, v. 38, p. 729 (Nov. 18, 1904).

VALUE OF THE RAILROADS AND THEIR CAPITALIZATION; by H. T. Newcomb. *Railroad Gazette*, v. 34, p. 671 (Aug. 29, 1902). (Argues that the capital stock of a railroad does not represent its real value; one page.)

VALUING RAILROAD PROPERTY; by Charles Hansel. *Traffic World*, v. 7, p. 735 (April 22, 1911). (Consideration of earning power, franchise and real value as elements of appraisal; address before the Southern Commercial Congress.)

WHAT IS THE VALUE OF A RAILROAD FOR PURPOSES OF TAXATION? by Charles Hansel. *Railroad Gazette*, v. 33, p. 271 (April 19, 1901). (On the work of Prof. M. E. Cooley, who was selected to examine the railroads in the State of Michigan, and on the subject of valuation in general; one page.)

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——Supplement. Government Printing Office, Washington, 1908.

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DEPRECIATION IN RAILWAY ACCOUNTING. (Letter); by H. A. Dunn. *Railway Age,* v. 45, p. 756 (May 29, 1908). (Comments on methods of accounting recommended by the Interstate Commerce Commission.)

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FORM OF GENERAL BALANCE SHEET STATEMENT AS PRESCRIBED BY THE Interstate Commerce Commission for Steam Roads. First Revised Issue, Effective June 15th, 1910. Government Printing Office, Washington, 1910.

HEARING ON DEPRECIATION OF EQUIPMENT ACCOUNTS BY INTERSTATE COMmerce Commission. *Electric Railway Journal,* v. 32, p. 236 (July 4, 1908). (Opinion of a Committee of the American Railway Association.)

——Editorial. Depreciation of Equipment Accounts. *Electric Railway Journal,* v. 32, p. 193 (July 4, 1908).

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United States Railway Valuation Act.

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——Bill for Physical Valuation of Railways. *Railway Age Gazette,* v. 52, p. 811 (April 5, 1912). (Full text of the Adamson Bill for the physical valuation of railways by the Interstate Commerce Commission.)

CAN ENGINEERS BE TRUSTED TO ARBITRATE FAIRLY AND INTELLIGENTLY Between the Public Interests and the Property Interests? (Editorial.) *Engineering News,* v. 69, p. 1187 (June 5, 1913.) (On statement of Senator Robert M. La Follette that railroad valuation work should be placed in charge of an economist and not of an engineer.)

HEARING ON PROPOSED FEDERAL RAILROAD APPRAISAL. *Railway and Engineering Review,* v. 53, p. 140 (Feb. 15, 1913).

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PLAN FOR PHYSICAL VALUATION OF RAILWAY PROPERTIES. *Railway Age,* v. 43, p. 286 (March 1, 1907). (Abstract of a memorandum by H. C. Adams submitted to the President by the Interstate Commerce Commission.)

THE PLANK IN THE DEMOCRATIC PLATFORM PLEDGING THE APPRAISAL OF All Railways. (Editorial.) *Engineering and Contracting,* v. 38, p. 562 (Nov. 20, 1912). (Estimates the cost of a railway appraisal to range from $2.50 per mile, for very rough inventory, to $25 per mile for a very thorough appraisal, or about 50 cents per $1 000 of physical property appraised.)

PROPOSED NATIONAL VALUATION CONVENTION. (Letter); by L. C. Fritch. *Railway Age Gazette,* v. 54, p. 1536 (June 20, 1913). (Objections to holding convention.)

RAILROADS—GENERAL—(*Continued*).

——The Railway Valuation Act. *Engineering News*, v. 69, p. 482 (March 6, 1913). (Text of the law passed by Congress ordering the valuation of railroad properties.)

THE RAILWAY VALUATION ACT. (Letter); by Alex. C. Humphreys. *Engineering News*, v. 69, p. 688 (April 3, 1913). (The doubtful value of the provision of the law which requires the appraisers to analyze past financial transactions of railroads.)

STATE RAILWAY COMMISSIONERS AND FEDERAL VALUATION; by D. F. Jurgensen. *Railway and Engineering Review*, v. 53, p. 529 (June 7, 1913). (States that the Federal valuation is a contest between carriers and public, with the Interstate Commerce Commission as umpire.)

VALUATION OF THE RAILROADS. (Editorial.) *Engineering Record*, v. 67, p. 283 (March 15, 1913). (On the Railway Valuation Act.)

THE VALUE OF VALUATION. (Editorial.) *Electric Railroad Journal*, v. 41, p. 667 (April 12, 1913). (Doubt expressed as to the usefulness of the valuation of railroads provided for by Congress.)

RAILROADS—SPECIAL CASES.

Atchison, Topeka & Santa Fé R. R.

WHY RAILROADS NEED HIGHER RATES, p. 88; by E. P. Ripley. Chicago (?), 1910. (Takes up the physical value of the Santa Fé R. R. in testimony before the Interstate Commerce Commission, Chicago, 1910.)

Beaumont & Great Northern R. R.

VALUATION OF THE RAILWAYS OF TEXAS. *Engineering-Contracting*, v. 33, p. 370 (April 20, 1910). (Detailed value of the Beaumont & Great Northern R. R., with brief comments.)

Boston & Maine R. R.

*EVIDENCE AS TO THE VALUE OF NEW HAMPSHIRE ROADS EMBRACED IN the Boston & Maine System. In the Report of the Public Service Commission of New Hampshire on an Investigation of Railroad Rates, p. 300. Concord, N. H., 1912. (Discussion of formula for determining the value of railroad properties: Original cost; amount and market value of stocks and bonds; present as compared with original cost of construction; probable earning capacity.)

California.

*INSTRUCTIONS TO ASSISTANT ENGINEERS FOR INSPECTING RAILWAY PROPerty for Inventory and Appraisal, California Railroad Commission. *Engineering and Contracting*, v. 37, p. 619 (May 29, 1912).

Chicago, Milwaukee & St. Paul R. R.

*A. E. BUELL VS. CHICAGO, MILWAUKEE & ST. PAUL RAILWAY COMPANY; Submitted July 1, 1906, Decided Feb. 16, 1907. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 1, pp. 337, 478. Madison, Wis., 1908. (Discusses what constitutes a fair valuation of a railroad.)

ITEMIZED COST OF THE C., M. & ST. P. R. R. IN SOUTH DAKOTA. *Engineering-Contracting*, v. 28, p. 56 (July 24, 1907). (Estimates given in the Spokane rate case.)

Great Britain.

THE SERVICEABLE LIFE AND COST OF RENEWALS OF PERMANENT WAY OF British Railways; by R. Price-Williams. *Journal*, Iron and Steel Institute, v. 80, p. 183 (1909, Pt. 2). (Gives actual figures and diagrams of cost of maintenance on British railways for a number of years.)

Great Northern Ry.

ITEMIZED COST OF THE GREAT NORTHERN RAILWAY SYSTEM AS ESTIMATED by Its Chief Engineer. *Engineering-Contracting*, v. 29, p. 271 (May 6, 1908). (Estimates given in testimony before the Interstate Commerce Commission in the Spokane rate case.)

ORIGINAL COST AND COST OF REPRODUCTION OF THE GREAT NORTHERN Railway (768 Miles) in the State of Washington. *Engineering-Contracting*, v. 32, p. 496 (Dec. 8, 1909). (Data as to the original cost, given by Halbert P. Gillette before the Railroad Commission.)

Kansas.

*KANSAS RAILWAY VALUATION, REPORT OF THE WORK OF THE ENGINEERing Department, Public Utilities Commission, from Date of Establishment to Nov., 1912; by C. C. Witt. *Public Service Regulation*, v. 2, p. 217 (May, 1913).

RAILROADS—SPECIAL CASES—(*Continued*).

Kansas City Southern Ry.

ACCOUNTING FOR ABANDONED PROPERTY. *Electric Railway Journal,* v. 39, p. 247 (Feb. 10, 1912). (Reply of Interstate Commerce Commission to the suit brought by the K. C. S. Ry., in regard to manner of accounting for value of property abandoned in the reconstruction of track.)

Louisiana.

***REPORT MADE BY THE STATE BOARD OF APPRAISERS SHOWING THE** Assessments Made of Property Employed in the Railway, Telegraph, Telephone, Sleeping Car, and Express Business for the Year 1899. (On the methods of valuation, and actual valuations made.)

Massachusetts.

THE FRANCHISE IN CAPITALIZATION. (Editorial.) *Railroad Gazette,* v. 37, p. 269 (Aug. 26, 1904). (Discussion of bill presented to Massachusetts Legislature allowing a new corporation to capitalize on a basis of market value of absorbed companies and to capitalize the franchises.)

Michigan.

EXPERT VALUATION OF RAILWAY AND OTHER CORPORATE PROPERTY IN Michigan. *Engineering News,* v. 44, p. 430 (Dec. 20, 1900). (Full account of the valuation made by the Michigan Board of State Tax Commissioners, describing organization for the work.)

RAILROAD TAXATION; by Robert H. Shields. In Sixth Report of the Board of State Tax Commissioners, Dec. 15, 1910, p. 53. Lansing, Mich., 1911. (Discusses several theories for valuation of railroads.)

RAILWAY CAPITAL AND VALUES; by W. H. Williams. *Railway Age Gazette,* v. 46, pp. 761, 805, 845, 903 (April 2, 9, 16, 23, 1909). (Outlines the methods used to obtain a physical valuation of railroad property in Michigan.)

——Abstract. *Railway and Engineering Review,* v. 48, p. 1047 (Dec. 26, 1908).

Minneapolis, St. Paul & Sault Ste. Marie Ry.

***IN RE INVESTIGATION ON MOTION OF THE COMMISSION OF PASSENGER** Rates Charged by the Minneapolis, St. Paul & Sault Ste. Marie Railway Company; Submitted April 9, 1907, Decided June 1, 1907. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 1, pp. 543, 581. Madison, Wis., 1908. (Contains brief reference to valuation of the road.)

Minnesota.

CURRENT RAILWAY PROBLEMS; by Samuel O. Dunn. Railway Age Gazette, New York, 1911. (Contains an article on valuation of railways with special reference to the physical valuation in Minnesota.)

***FORMS USED IN COMPILING INFORMATION IN THE 1906 APPRAISAL OF THE** Railways of Minnesota. *Engineering and Contracting,* v. 37, p. 52 (Jan. 10, 1912). (Four pages.)

JUDGE SANBORN ON STATE INTERFERENCE WITH INTERSTATE COMMERCE and Valuation of Railways. *Railway Age Gazette,* v. 50, p. 987 (April 28, 1911). (Opinions of the U. S. Circuit Judge in the Minnesota rate case; abstract of the discussion on the correct basis for the valuation of railways and a "fair return")

THE MINNESOTA COMMISSION'S VALUATION FORMS. *Public Service Regulation,* v. 1, p. 301 (May, 1912). (Showing headings of some of the blanks used for detailed railway inventory and valuation in Minnesota, contributed by D. F. Jurgensen, Engineer.)

THE MINNESOTA RATE CASE; by Charles E. Otis. In Railway Library, 1910, p. 31; edited by Slason Thompson. Bureau of Railway News and Statistics, Chicago, 1911. (Contains analysis of valuation of property devoted to public service.)

THE MINNESOTA RATE CASES, OPINION OF THE SUPREME COURT OF THE United States. (63d Congress, 1st Sess., Senate Doc. No. 54). Washington, D. C., 1913. (Opinion of Justice Hughes of the Supreme Court on the valuation of railways.)

——Abstracts. Decision of the Supreme Court in the Minnesota Rate Cases. *Electric Railway Journal,* v. 41, p. 1064 (June 14, 1913); Railway Evaluation and Depreciation. *Engineering Record,* v. 67, p. 687 (June 21, 1913); The Supreme Court's Comments on Railway Valuation. *Railway Age Gazette,* v. 54, p. 1537 (June 20, 1913).

——Editorials. Construction Expenses in the Minnesota Rate Decision. *Electric Railway Journal,* v. 41, p. 1101 (June 21, 1913); *Railway Age Gazette,* v. 54, p. 1304 (June 13, 1913); The Minnesota Rate Decision. *Engineering Record,* v. 67, p. 679 (June 21, 1913); The United States Supreme Court on Principles of Railway Valuation. *Engineering News,* v. 69, p. 1338 (June 26, 1913).

RAILROADS—SPECIAL CASES—(*Continued*).

MINNESOTA RATE LAWS VOID. *Traffic World,* v. 7, p. 646 (April 15, 1911). (Statement is made that apportionment on the basis of revenue to the various classes of its business in order to determine the reasonableness of its rate, is the most reaonable and equitable method of assigning the value of a railroad property in a State, used for transportation; decision in case Shepard *vs.* Northern Pacific *et al.,* etc., United States Circuit Court.)

*TWENTY-THIRD ANNUAL REPORT OF THE RAILROAD AND WAREHOUSE Commission of Minnesota to the Governor for the Year Ending November 30, 1907, p. 14. St. Paul, 1908. (On method of making valuation; six pages.)

*——24th, 1908, p. 12. Minneapolis, 1909. (Contains report on valuation of the railways of the State.)

——Abstracts. An Analysis of the Appraisal of the Railways of Minnesota with Comments on the Same. *Engineering-Contracting,* v. 31, p. 172 (March 3, 1909); Valuation of Railways in Minnesota. *Railway Age Gazette,* v. 46, p. 225 (Jan. 29, 1909).

*——26th, 1910, p. 94. Minneapolis, 1911. (Contains findings and report on rate cases by Charles E. Otis, including valuations of property devoted to public service.)

——Abstract. *Railway and Engineering Review,* v. 49, p. 297 (March 13, 1909).

VALUATION OF RAILROAD PROPERTY IN MINNESOTA; by A. S. Cutler. *Year Book,* Engineers' Society, University of Minnesota, v. 16, p. 69 (1908). (Method of making valuation of railroad property in Minnesota; seven and one-half pages.)

VALUATION OF RAILWAYS IN MINNESOTA. *Railway Age Gazette,* v. 44, p. 877 (Dec. 20, 1907). (Describes methods of making valuation.)

VALUATION OF RAILWAYS IN MINNESOTA. *Railway Age Gazette,* v. 46, p. 269 (Feb. 5, 1909). (Describes the work of the Minnesota Railroad Commission.)

——Editorial. *Railway Age Gazette,* v. 46, p. 245 (Feb. 5, 1909).

Nebraska.

*FOURTH ANNUAL REPORT OF THE NEBRASKA STATE RAILWAY COMMISsion to the Governor, Year Ending November 30, 1911, p. 446. Omaha, 1911. (Includes statements of physical value of the various railroad and transportation facilities of the stock yards properties within the State of Nebraska, being the initial appraisal of July 1st, 1909.)

——State Railway Valuation: Nebraska Commission Reports Progress on State-Wide Appraisal of Physical Values. *Public Service Regulation,* v. 1, p. 445 (July, 1912).

ORGANIZATION FOR AND METHODS AND RESULTS OF PHYSICAL VALUATION in Nebraska; by E. C. Hurd. *Engineering and Contracting,* v. 36, p. 694 (Dec. 27, 1911). (Two pages.)

VALUATION OF RAILWAY PROPERTY—THE NEBRASKA METHOD. *Railway Age,* v. 37, p. 1219 (June 24, 1904). (Valuation for purposes of taxation; one page.)

New Jersey.

THE APPRAISAL OF THE RAILWAYS OF NEW JERSEY, AND PROGRESS IN Other States. *Engineering and Contracting,* v. 35, p. 701 (June 21, 1911). (One column.)

*PROGRESS REPORT OF THE BOARD TO RE-APPRAISE RAILROADS AND Canals in This State, March 9th, 1910. Trenton, N. J. (Gives organization, methods, and some data on actual values.)

RAILROAD APPRAISAL AND TAXATION IN NEW JERSEY; by Charles Hansel. *Engineering News,* v. 68, p. 334 (Aug. 22, 1912). (Defense of the method of appraisal of railroads for taxation in New Jersey.)

*REPORT ON REVALUATION OF RAILROADS AND CANALS, NEW JERSEY, 1911; by Charles Hansel. Trenton, 1912. (Treats of value of property, including the franchise, bonds and stocks, organization, operating cost, real estate, tangible personal property, depreciation, etc.)

THE VALUATION OF RAILROADS IN NEW JERSEY; by Charles Hansel. *Engineering Record,* v. 63, p. 594 (May 27, 1911). (Four pages.)

VALUATION OF RAILWAYS IN NEW JERSEY. *Railway Age Gazette,* v. 53, p. 243 (Aug. 9, 1912). (Methods adopted by Charles Hansel to ascertain value in accordance with State statute; results obtained by the work.)

New York, New Haven & Hartford R. R.

DEPRECIATION FUNDS OF THE NEW YORK, NEW HAVEN AND HARTFORD Railroad. (Editorial.) *Railway Age,* v. 43, p. 133 (Feb. 1, 1907). (One paragraph.)

RAILROADS—SPECIAL CASES—(*Continued*).

THE NEW HAVEN VALUATION. *Railway Age Gazette,* v. 50, p. 461 (March 10, 1911). (Review of the valuation of the N. Y., N. H. & H. R. R. properties just completed under the order of the Massachusetts Legislature.)

PRICES EMPLOYED IN THE PHYSICAL VALUATION OF THE NEW YORK, NEW Haven & Hartford R. R. *Engineering and Contracting,* v. 37, p. 220 (Feb. 21, 1912). (The unit prices adopted were based on the average ruling prices of the various elements for the last few years and on prices actually paid by the Railway Company; one column.)

THE PRINCIPLES GOVERNING A RAILROAD APPRAISAL OF AN UNUSUAL NAture. *Engineering Record,* v. 65, p. 174 (Feb. 17, 1912). (Gives an outline of the views of George F. Swain on the valuation of the N. Y., N. H. & H. R. R.)

RAILROAD REVALUATION—WITH AN EXAMPLE. (Editorial.) *Railway Age Gazette,* v. 45, p. 1081 (Oct. 9, 1908). (On the valuation of the N. Y., N. H. & H. R. R.)

——Letter. Scientific Valuation in Wisconsin; by Dwight C. Morgan. *Railway Age Gazette,* v. 45, p. 1242 (Oct. 30, 1908). (Criticism of editorial of Oct. 9, 1908.)

*REPORT OF THE BOARD OF RAILROAD COMMISSIONERS, THE TAX COMMISsioner and the Bank Commissioner, Sitting as a Commission, Relative to the Assets and Liabilities of the New York, New Haven & Hartford Railroad Company. Wright & Potter Printing Co., Boston, 1911. (Contains extensive reports by George F. Swain and Stone & Webster on the valuation of the railroad.)

VALUATION OF THE SOUTH STATION, BOSTON. (Editorial.) *Railway Age Gazette,* v. 48, p. 1243 (May 20, 1910). (On the revaluation of the South Station at Boston, made by J. F. Stevens, for the N. Y., N. H. & H. R. R. Co.; one-half column.)

Northern Pacific Ry.

AVERAGE COST OF REPAIRING LOCOMOTIVES IN AMERICA, COMPARED WITH the Cost on the Northern Pacific, Together with Comments on Plant Depreciation and Repairs. *Engineering-Contracting,* v. 30, p. 150 (Sept. 2, 1908). (Discusses depreciation of locomotives.)

FINDINGS OF THE INTERSTATE COMMERCE COMMISSION AS TO THE COSTS of Constructing the Northern Pacific and Great Northern Railways, and Its Decision in the Spokane Rate Case. *Engineering-Contracting,* v. 31, p. 217 (March 24, 1909). (Discusses a decision of the Interstate Commerce Commission regarding rate reduction on the N. P. and G. N. Rys.)

ITEMIZED COST OF THE NORTHERN PACIFIC RAILWAY SYSTEM AS ESTImated by Its Chief Engineer. *Engineering-Contracting,* v. 29, p. 226 (April 15, 1908). (Estimates given in the Spokane rate case.)

*ORIGINAL COST AND COST OF REPRODUCTION OF THE NORTHERN PACIFIC Railway (1 645 Miles) in the State of Washington. *Engineering-Contracting,* v. 33, p. 44 (Jan. 12, 1910). (Data as to the cost given by Halbert P. Gillette, before the Railroad Commission.)

See also Minnesota.

Oklahoma.

*BEFORE THE CORPORATION COMMISSION, STATE OF OKLAHOMA: *In Re* Proposed Order to Promulgate Rates on Wheat, etc., No. 1350. Oklahoma City, 1912. (Opinion of George A. Henshaw, Commissioner; contains an extended discussion of the proper elements in valuation.)

Oregon.

*ANNUAL REPORT OF THE RAILROAD COMMISSION OF OREGON, 1908, p. 20; 1909, p. 25. Salem, Ore., 1909-10. (Speaks briefly of the methods used and describes the organization of the force.)

Oregon R. R. & Navigation Co.

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POLICY OF ENGLISH MUNICIPAL TRAMWAYS RESPECTING RENEWALS. *Electric Railway Journal,* v. 38, p. 661 (Oct. 7, 1911). (Abstract of Finance and Policy, by James H. Rodgers; paper read before the Municipal Tramways Association.)

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——Abstract. *Electric Railway Journal,* v. 38, p. 766 (Oct. 12, 1911).

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ROLLING STOCK DEPRECIATION. (Editorial.) *Street Railway Journal,* v. 30, p. 240 (Aug. 17, 1907).

SOME FUNDAMENTAL CONSIDERATIONS IN DEPRECIATION. (Editorial.) *Electric Railway Review,* v. 17, p. 144 (Feb. 2, 1907). (Method of accounting for depreciation.)

STREET AND INTERURBAN RAILROADS—GENERAL—(*Continued*).

SPECIAL REPORTS, STREET AND ELECTRIC RAILWAYS, 1907, p. 165. United States Bureau of the Census. Washington, 1910. (Definition, object and methods allowing for depreciation in electric railway property and estimated per cent. for depreciation allowed by the Chicago Union Traction Co., the Third Avenue R. R. of New York, and the Milwaukee Electric Ry. & Light Co.)

STREET RAILWAY ACCOUNTING; by A. S. Michenner. *Stone and Webster Public Service Journal,* v. 3, p. 92 (Aug., 1908). (Paper read before the Massachusetts Institute of Technology; refers to classification of accounts, including allowance for depreciation.)

A THEORETICAL BASIS FOR DETERMINING FARES ON PROPERTIES HAVING Annual Gross Earnings of from $100 000 to $5 000 000; by C. N. Duffy. *Proceedings,* American Electric Railway Association, 1912, p. 246. (Data on cost of providing the service.)

——Abstract. *Electric Railway Journal,* v. 40, p. 1103 (Nov. 30, 1912).

TRAMWAY DEPRECIATION; by A. J. J. Pfeiffer. *Tramway and Railway World,* v. 28, p. 61 (Aug. 4, 1910). (A discussion of the subject from the viewpoint of conditions in Great Britain; theoretical.)

TRAMWAYS DEPRECIATION. (Editorial.) *Municipal Journal* (London), v. 11, p. 287 (April 4, 1902). (Discussion of necessary allowance for depreciation in street railways.)

TREATMENT OF DEPRECIATION OF STREET RAILWAY PROPERTIES; by Frank R. Ford. *Engineering-Contracting,* v. 38, p. 560 (June 15, 1910). (From a paper read before the American Street and Interurban Railway Association, January, 1910; three pages.)

——*Electric Railway Journal,* v. 35, p. 284 (Feb. 12, 1910).

***UNIFORM CLASSIFICATION OF ACCOUNTS FOR ELECTRIC RAILWAYS, PRE-** scribed by the Railroad Commission of Wisconsin, January, 1909. Edition 2. Madison, Wis., 1912. (Includes treatment of reserve accounts, depreciation, sinking fund and amortization.)

VALUATION AND RATES. (Editorial.) *Street Railway Journal,* v. 30, p. 307 (Aug. 31, 1907). (On the similarity of street and steam railroad valuation; one and one-half columns.)

VALUATION OF A SHORT-TERM FRANCHISE. (Editorial.) *Electric Railway Review,* v. 17, p. 313 (March 9, 1907). (Should reconstruction and renewals be charged to capital accounts or provided for out of earnings, etc.)

VALUATION OF INTANGIBLE STREET RAILWAY PROPERTY; by Frank R. Ford. In "Electric Railway Transportation," p. 119. *Annals,* American Academy of Political and Social Science, v. 37, No. 1 (Jan., 1911). (Twenty-two pages.)

WHERE MAINTENANCE ENDS AND DEPRECIATION BEGINS; by J. H. Neal. *Proceedings,* American Street and Interurban Railway Accountants' Association, 1907, p. 195. (Discusses the relation between maintenance and depreciation.)

——*Street Railway Journal,* v. 30, p. 700 (Oct. 19, 1907).

WORK OF VALUATION OF ELECTRIC RAILWAY PROPERTY; by H. R. Ralph Badger. *Electric Traction Weekly,* v. 6, p. 197 (Feb. 19, 1910). (Methods of determining physical and intangible values.)

United States Interstate Commerce Commission.

ACCOUNTING CIRCULAR OF THE INTERSTATE COMMERCE COMMISSION. *Electric Railway Review,* v. 19, p. 266 (Feb. 29, 1908). (Classification of accounts for electric railways, including depreciation accounts, from Circular No. 20 of the U. S. Interstate Commerce Commission.)

ACCOUNTING FOR DEPRECIATION AS PRESCRIBED BY THE INTERSTATE COM-merce Commission. (Editorial.) *Electric Railway Review,* v. 18, p. 32 (July 13, 1907). (Comments on paper by H. J. Davies, who outlined a method of providing for depreciation or renewal reserves for an electric railway.)

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THE INTERSTATE COMMERCE CLASSIFICATION. (Letter); by H. M. Kochersperger. *Street Railway Journal,* v. 31, p. 729 (May 2, 1908). (On the inapplicability of the classification to electric railways.)

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STREET AND INTERURBAN RAILROADS—GENERAL—(*Continued*).

THE INTERSTATE COMMERCE CLASSIFICATION OF ACCOUNTS. (Letter); by W. W. May. *Street Railway Journal*, v. 31, p. 613 (April 11, 1908). (Objections to the system of accounting prescribed in Accounting Circular No. 20, U. S. Interstate Commerce Commission.)

MILWAUKEE ELECTRIC RAILWAY & LIGHT COMPANY'S REPLY TO ACCOUNTing Circular; by C. N. Duffy. *Electric Railway Journal*, v. 19, p. 419 (April 4, 1908). (Reply to Accounting Series Circular No. 20 of the U. S. Interstate Commerce Commission.)

REPORT ON INTERSTATE ACCOUNTING SYSTEM; by H. E. Adams. *Electric Railway Journal*, v. 34, p. 218 (Aug. 7, 1909). (Statement regarding the classification of accounts of electric railways required by the Interstate Commerce Commission with paragraphs on depreciation and valuation.)

REVISED CLASSIFICATION OF ACCOUNTS FOR ELECTRIC RAILWAYS. *Electric Railway Review*, v. 19, p. 624 (May 23, 1908). (Classification of the Inter-State Commerce Commission, effective as of Oct. 1, 1908.)

SUIT AGAINST CLASSIFICATION OF ADDITIONS AND BETTERMENTS OF THE Interstate Commerce Commission. *Electric Railway Journal*, v. 38, p. 1067 (Nov. 18, 1911). (One-half column.)

STREET AND INTERURBAN RAILROADS—SPECIAL CASES.

Augusta-Aiken Ry. & Electric Co.

INSURANCE FUND AND DEPRECIATION RESERVES. (Letter); by John Blair MacAfee. *Street Railway Review*, v. 15, p. 292 (May 15, 1905). (Plans of the Augusta-Aiken Ry. & Electric Co. for taking care of depreciation.)

Boston Elevated Ry.

HEARING ON ELECTRIC RAILWAY MAIL PAY. *Electric Railway Journal*, v. 41, p. 291 (Feb. 15, 1913). (Details of estimated cost of operation of present type of mail car on Boston Elevated Ry.; car, power, track investment, etc.; includes per cent. allowed for depreciation in each case.)

Brooklyn, N. Y.

ALLOWANCE FOR OBSOLESCENCE UPHELD IN FRANCHISE TAX CASE. *Electric Railway Journal*, v. 36, p. 1154 (Dec. 10, 1910). (Decision of New York Supreme Court in favor of Brooklyn Rapid Transit Co.; one-half page.)

APPROXIMATE VALUE PLACED ON PHYSICAL PROPERTY OF BROOKLYN Transit System. *Electric Railway Journal*, v. 34, p. 1261; v. 35, pp. 156, 248 (Dec. 25, 1909; Jan. 22, Feb. 5, 1910). (Testimony of B. J. Arnold and T. S. Williams before New York Public Service Commission.)

HEARING ON VALUATION OF CONEY ISLAND AND BROOKLYN R. R. *Electric Railway Journal*, v. 34, pp. 377, 398, 437, 469, 878, 1108, 1148, 1188, 1263; v. 35, pp. 104, 460 (Sept. 4, 11, 18, 25, Oct. 16, Nov. 27, Dec. 4, 11, 25, 1909; Jan. 15, March 12, 1910). (Hearing before the New York Public Service Commission, First District.)

MAY RESERVE FUND TO RENEW OBSOLETE EQUIPMENT. *Electric Traction Weekly*, v. 6, p. 1473 (Dec. 31, 1910). (Decisions of the Supreme Court at Albany that there may be a deduction for obsolescence as distinguished from depreciation in connection with the valuation of special franchise of the Brooklyn Rapid Transit Co.)

*RE MacREYNOLDS V. BROOKLYN UNION ELEVATED RAILROAD COMPANY (Case 353). Reports and Decisions of the Public Service Commission, First District of the State of New York, v. 2, p. 246. New York, 1912. (Relation of fares and the valuation of Brooklyn Union Elevated R. R. Co.)

*RE MONHEIMER V. CONEY ISLAND & BROOKLYN RAILROAD COMPANY (Case 350). Reports and Decisions of the Public Service Commisison, First District of the State of New York, v. 1 p. 705. New York, 1912. (Valuation of the Coney Island & Brooklyn R. R. Co. in relation to fares.)

*——A Ten-cent Fare to Coney Island Upheld by Public Service Commission. *Electric Railway Journal*, v. 35, p. 456 (March 12, 1912). (Decision of Commission after considering the testimony as to the value of the Brooklyn Rapid Transit Co.)

THEORY OF STREET RAILWAY RATE REGULATION AS DEVELOPED IN THE Coney Island Fare Case; by Frank R. Ford. *Proceedings*, American Street and Interurban Railway Association, v. 29, p. 159 (1910). (On valuation; twelve pages.)

——*Electric Railway Journal*, v. 36, pp. 712, 752 (Oct. 12, 1910).

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

Buffalo, N. Y.

DEPRECIATION CHARGES OF THE INTERNATIONAL TRACTION COMPANY. (Editorial.) *Electric Railway Review,* v. 19, p. 472 (April 18, 1908). (The definite policy of the International Traction Co. of Buffalo, in accounting currently for depreciation.)

FURTHER TESTIMONY IN BUFFALO REORGANIZATION CASE; by F. A. Sager. *Electric Railway Journal,* v. 39, p. 246 (Feb. 10, 1912). (Reviews methods followed in the inventory of the physical property of the International Traction Co.)

HEARING IN BUFFALO ON INTERNATIONAL TRACTION REORGANIZATION. *Electric Railway Journal,* v. 38, pp. 910, 991 (Oct. 21, Nov. 4, 1911). (An account of the methods used by B. J. Arnold in determining the cost to produce new the physical property of the International Traction Co. of Buffalo.)

——Editorial. Values Claimed by the Buffalo Plan. *Electric Railway Journal,* v. 38, p. 976 (Nov. 4, 1911).

California.

*ORDER OF CALIFORNIA COMMISSION REQUIRING VALUATIONS OF ELECTRIC Railways. *Electric Railway Journal,* v. 40, p. 1027 (Nov. 16, 1912).

Cardiff, Wales.

CARDIFF TRAMWAYS FINANCE. *Electrical Engineer,* v. 44, p. 879 (Dec. 24, 1909). (Discusses depreciation of electric railway at Cardiff.)

DEPRECIATION, INTERESTING REPORT FROM CARDIFF. *Municipal Journal* (London), v. 16 p. 1083 (Dec. 20, 1907). (Allowance for depreciation considered to represent fair wear and tear.)

DEPRECIATION OF CARDIFF ELECTRIC TRAMWAY AND LIGHTING UNDERTAKings. *Electric Railway Review,* v. 19, p. 16 (Jan. 4, 1908). (Abstract from *Electrical Engineer* (London) giving details of depreciation of equipment.)

DEPRECIATION OF CARDIFF PROPERTIES. *Electric Railway Journal,* v. 36, p. 409 (Sept. 10, 1910). (Brief report on the street railway of Cardiff in relation to depreciation.)

TRACK DEPRECIATION AT CARDIFF; by John Allcock. *Tramway and Railway World,* v. 27, p. 30 (Jan. 6, 1910). (Brief statement.)

Chicago, Ill.

THE BASIS OF VALUATION IN THE CASE OF MUNICIPAL PURCHASE OF STREET Railways; by Sidney Ossoski. *Electric Railway Journal,* v. 36, p. 999 (Nov. 12, 1910). (Discusses methods of valuation using street railways of Chicago and Cleveland as examples; two pages.)

CHICAGO ELEVATED RAILWAY VALUATION. *Electric Railway Journal,* v. 39, p. 1087 (June 22, 1912). (Elements entering into the value of each piece of property in real estate valuation.)

——Abstracts. Reports of Appraisal of the Physical Properties of the Elevated Railways of Chicago. *Engineering and Contracting,* v. 37 (May 15, 1912); The Valuation of the Elevated Railroads of Chicago. *Engineering Record,* v. 65, p. 552 (May 18, 1912).

——Editorial. *Engineering Record,* v. 65, p. 534 (May 18, 1912). (Very brief.)

CHICAGO ELEVATED RAILWAYS, REPORT ON VALUATION OF PHYSICAL Property Including Real Estate and Rights of Way of the South Side Elevated Railroad Company, Metropolitan West Side Elevated Railway Company, Northwestern Elevated Railroad Company and Chicago & Oak Park Elevated Railroad Company, to the Local Transportation Committee of the City Council of Chicago, April 30, 1912, Reprinted May 9, 1912, with the Addition of the Final Figures of the Valuation Commission; by George F. Swain. Chicago, 1912.

CHICAGO ELEVATED RAILWAYS VALUATION. *Electric Railway Journal,* v. 39, p. 919 (June 1, 1912). (Analysis of right-of-way values.)

CHICAGO VALUATIONS — AGREEMENTS TO TERMS BY COMPANIES. *Electric Railway Journal,* v. 28, p. 1164 (Dec. 22, 1906). (Abstract of the report of B. J. Arnold, M. E. Cooley and A. B. du Pont, on valuation of the Chicago City Ry. Co. and the Chicago Union Traction Co.)

DEPRECIATION AND CHICAGO VALUATION FIGURES. (Editorial Correspondence.) *Electric Traction Weekly,* v. 8, p. 584 (May 18, 1912).

DETAILED EXHIBITS OF THE TANGIBLE PROPERTY OF THE CHICAGO CITY Railway Company as of June 30, A. D. 1906, Accompanying the Valuation Report Submitted to the Committee on Local Transportation of the Chicago City Council; by Bion J. Arnold, Mortimer E. Cooley and A. B. du Pont. Chicago, 1906.

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

DETAILED EXHIBITS OF THE TANGIBLE PROPERTY OF THE STREET RAILWAY System in the Possession of and Operated by the Receivers of the Chicago Union Traction Company as of June 30, A. D. 1906, Accompanying the Valuation Report Submitted to the Committee on Local Transportation of the Chicago City Council; by Bion J. Arnold, Mortimer E. Cooley and A. B. du Pont. Chicago, 1906.

ELEMENTS OF VALUE IN A STREET RAILWAY. *Railroad Gazette,* v. 41, p. 567 (Dec. 28, 1906). (Valuation of the Chicago street railways by a commission prior to their purchase by the city; one page.)

ELEVATED VALUES IN CHICAGO. (Editorial.) *Electric Railway Journal,* v. 39, p. 817 (May 18, 1912).

ESTIMATED COST OF CABLE RAILWAYS IN CHICAGO. *Engineering and Contracting,* v. 37, p. 338 (March 20, 1912). (Cost of reproducing cable railway new as estimated by Bion J. Arnold.)

ITEMIZED UNIT COSTS OF 98 SPECIAL OVERHEAD LAYOUTS FOR A TROLLEY Railway. *Engineering-Contracting,* v. 34, p. 335 (Oct. 19, 1910). (Inventory made by the Traction Valuation Commission of Chicago.)

MAINTENANCE AND DEPRECIATION CHARGES OF THE CHICAGO UNION TRAC-tion Company. *Electric Railway Review,* v. 17, p. 247 (Feb. 23, 1907). (The policy of the City of Chicago, in allowing for depreciation in street railway property.)

——Editorial. Amount of Maintenance and Depreciation Charges. *Electric Railway Review,* v. 17, p. 244 (Feb. 23, 1907).

METHODS OF CONDUCTING THE VALUATION OF THE PHYSICAL PROPERTIES of the Chicago Consolidated Traction Co., with Summaries of Costs; by Philip J. Kealy. *Engineering-Contracting,* v. 34, pp. 274, 295 (Sept. 28, Oct. 5, 1910). (The valuation covers only that portion of the system within the city limits; describes methods and costs of track and power-house valuation and data of the electric power distribution.)

OPERATIONS OF THE COMPANIES UNDER THE 1907 ORDINANCES (CHICAGO). *Electric Railway Journal,* v. 40, p. 525 (Oct. 5, 1912). (Comparative values of street railways of Chicago, Commission's and Companies' estimates.)

RENEWALS AS DEFINED BY THE BOARD OF SUPERVISING ENGINEERS, CHI-cago Traction. *Electric Railway Journal,* v. 37, p. 374 (March 4, 1911). (Classification of renewals of track, equipment, buildings and bridges.)

REPORT ON THE ENGINEERING AND OPERATING FEATURES OF THE CHICAGO Transportation Problem, Submitted to the Committee on Local Transportation of the Chicago City Council, p. 182; by Bion Joseph Arnold. Chicago, 1902. (Gives unit price estimates, valuation estimates, valuations under expiring grants, and cost of estimates; fifty-five pages.)

REPORT ON THE PHYSICAL PROPERTIES AND INTANGIBLE VALUES OF THE Calumet Electric Street Railway Company and the South Chicago City Railway Company as of February 1, A. D. 1908, Submitted to the Committee on Local Transportation of the Chicago City Council; by Bion J. Arnold. Chicago, 1908. 3 v. (General summary of value of physical property and detailed exhibits.)

STREET RAILWAY APPRAISAL METHODS AT CHICAGO. (Editorial.) *Engineering Record,* v. 62, p. 501 (Oct. 29, 1910). (Four columns.)

TWO REPORTS SUBMITTED TO COUNCIL COMMITTEE ON VALUES OF CHICAGO Elevated Railways. *Electric Railway Journal,* v. 39, p. 797 (May 11, 1912). (Comparison of reports by George F. Swain and the one submitted by the Harbor and Subway Commission.)

UNIT PRICES USED IN THE FIRST APPRAISAL OF ELECTRIC RAILWAYS IN Chicago. *Engineering and Contracting,* v. 37, p. 393 (April 3, 1912). (Detailed estimate of cost of street railway property in Chicago.)

VALUATION OF THE PROPERTY OF THE CHICAGO CONSOLIDATED TRACTION Co.; by B. J. Arnold and George W. Weston. *Engineering News,* v. 64, p. 241 (Sept. 1, 1910). (Short paragraph.)

VALUATION OF TWO STREET RAILWAY POWER PLANTS. *Engineering-Contracting,* v. 34, p. 280 (Sept. 28, 1910). (Part of the property of the Chicago Consolidated Traction Co.; two pages.)

VALUATION REPORTS ON CHICAGO ELEVATED ROADS. (Editorial Correspondence.) *Electric Traction Weekly,* v. 8, p. 556 (May 11, 1912).

VALUATIONS OF CHICAGO ELEVATED RAILWAYS. *Electric Railway Journal,* v. 39, p. 829 (May 18, 1912). (Detail figures and summaries of the explanatory statements made in connection with the two valuations.)

VALUE OF PROPERTY OF THE CHICAGO CONSOLIDATED TRACTION COM-pany. *Electric Railway Journal,* v. 36, pp. 309, 374, 1111 (Aug. 20, Sept. 3, Dec. 3, 1910). (Valuation made by Bion J. Arnold and George W. Weston.)

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

Cleveland, Ohio.

ARBITRATION OF OPERATING EXPENSE CHARGES IN CLEVELAND. *Electric Railway Journal,* v. 41, p. 925 (May 24, 1913). (Includes brief reference to depreciation of the railway plant and maintenance, renewal and depreciation fund.)

DECISION OF ARBITRATOR IN CLEVELAND CONTROVERSY. *Electric Railway Journal,* v. 34, p. 1237 (Dec. 25, 1909). (Findings of Judge Taylor in the Cleveland Street Railway valuation.)

DECISION OF BOARD OF ARBITRATION IN THE CLEVELAND CASE. *Electric Railway Journal,* v. 41, p. 1159 (June 28, 1913). (Text of the finding, including allowance for maintenance, renewal and depreciation.)

——Editorial. The Arbitrated Result in Cleveland. *Electric Railway Journal,* v. 41, p. 1134 (June 28, 1913).

DEPRECIATION IN CLEVELAND. *Street Railway Journal,* v. 29, p. 743 (April 27, 1907). (On depreciation of track and cars; abstract from report of the Cleveland Electric Ry. Co.)

FINAL TESTIMONY AND ARGUMENTS IN THE CLEVELAND CASE. *Electric Railway Journal,* v. 41, p. 1070 (June 14, 1913). (Depreciation in the value of the property shown by tables submitted by Henry J. Davies.)

MAINTENANCE PROVISIONS OF CLEVELAND ORDINANCE; by H. J. Davies. *Electric Railway Journal,* v. 35, p. 614 (April 2, 1910). (Provisions for maintenance of physical property of Cleveland Ry. Co. determined in granting new franchise ordinance.)

OPERATION OF THE CLEVELAND STREET RAILWAY SYSTEM BY A NEW COMpany. *Electric Railway Journal,* v. 32, p. 433 (Aug. 8, 1908). (Provisions for maintenance and renewal fund in lease.)

TESTIMONY IN CLEVELAND VALUATION. *Electric Railway Journal,* v. 34, pp. 1024, 1068, 1159 (Nov. 13, 20, Dec. 4, 1909). (Opinions on valuation of the Cleveland Street Railway given by Frank R. Ford, Bion J. Arnold and others before Judge Taylor of the United States Circuit Court.)

VALUATION OF THE CLEVELAND ELECTRIC RAILWAY. *Electric Railway Review,* v. 19, p. 149 (Feb. 1, 1908). (Values of physical property, overhead charges, franchises, etc.; two pages.)

Detroit, Mich.

APPRAISAL OF THE CITY LINES OF THE DETROIT UNITED RAILWAY. *Electric Railway Journal,* v. 41, p. 897 (May 17, 1913). (Methods and summary of valuation.)

REPORT AND APPRAISAL OF THE DETROIT UNITED RAILWAY (CITY LINES), Detroit, Michigan, Oct. 1, 1909; by Frederick T. Barcroft. Detroit, 1910. (Contains brief data on method of making appraisal.)

——Abstract. The Appraisal Value of the Electric Street Railways of Detroit, Mich. *Engineering-Contracting,* v. 34, pp. 16, 35 (July 6, 13, 1910).

——Editorial. Noteworthy Article on Electric Street Railway Appraisal. *Engineering-Contracting,* v. 34, p. 1 (July 6, 1910).

REPORT OF THE COMMITTEE OF FIFTY. *Electric Railway Journal,* v. 36, pp. 111, 142 (July 16, 23, 1910). (Abstract of report of Committee to investigate Detroit Street Railway situation; contains brief references to the appraisal.)

REPORTS ON VALUATION OF DETROIT PROPERTY. *Electric Railway Journal,* v. 34, p. 1077 (Nov. 20, 1909). (Brief comparison of the valuations made by F. T. Barcroft and R. B. Rifenberick.)

RESULTS OF DETROIT INVESTIGATION. *Electric Railway Journal,* v. 34, p. 1276 (Dec. 25, 1909). (Concerning reports received by the Committee of Fifty on the street railway valuation.)

A STATEMENT OF "FACTS" CONCERNING THE SO-CALLED "BARCROFT Appraisal" of the Detroit United Railway Lines in the City of Detroit; by R. B. Rifenberick. Detroit, Mich., 1910. (A criticism of Mr. Barcroft's methods.)

THE VALUATION OF THE DETROIT STREET RAILWAYS. *Engineering News,* v. 64, p. 212 (Aug. 25, 1910). (An explanation of the situation with comparison of valuations made for the City with those for the Company; two pages.)

VALUATION OF THE TRACK OF THE DETROIT STREET RAILWAY SYSTEM. *Engineering News,* v. 64, p. 249 (Sept. 8, 1910). (Explains methods used in track valuation, including estimated value of twenty-one types of rail sections; one page.)

VALUATIONS OF THE DETROIT UNITED RAILWAY. *Electric Railway Journal,* v. 36, pp. 258, 294 (Aug. 13, 20, 1910). (Review of the facts, statement of the position of the Company regarding the Barcroft appraisal, brief abstract of the Barcroft appraisal, and a statement by R. B. Rifenberick.)

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

Duluth, Minn.

DECISION OF WISCONSIN COMMISSION IN SUPERIOR CASES. *Electric Railway Journal,* v. 40, p. 1067 (Nov. 23, 1912). (Data on valuation of property of Duluth Street Ry. Co.)

Eastern Ry. & Light Co. See Fond du Lac, Wis.

Europe.

DEPRECIATION AS APPLICABLE TO ELECTRIC RAILWAYS; by Haselmann. *Electric Railway Journal,* v. 28, p. 1003 (Nov. 24, 1906). (Abstract of a paper read before the International Street and Interurban Railway Association giving methods of depreciation accounting in Europe; three pages.)

DEPRECIATION FUNDS IN EUROPE. *Electric Railway Journal,* v. 23, p. 696 (May 7, 1904). (Gives allowances made for depreciation.)

——Editorial. Depreciation. *Electric Railway Journal,* v. 23, p. 760 (May 21, 1904).

Fond du Lac, Wis.

***EXISTING FARES OF WISCONSIN ROAD UPHELD BY COMMISSION AFTER A** Valuation. *Electric Railway Journal,* v. 38, p. 193 (July 9, 1911). (Valuation of electric railway properties and division of valuation between city and interurban systems; decision of the Railroad Commission of Wisconsin in a fare case involving the Eastern Ry. & Light Co. of Fond du Lac.)

Fonda, Johnstown & Gloversville R. R.

LIFE OF RAILWAY PHYSICAL PROPERTY FROM THE ENGINEERING STAND-point; by F. A. Bagg. *Electric Railway Journal,* v. 38, p. 1205 (Dec. 9, 1911). (Paper read before Street Railway Association of State of New York; discusses life of track and overhead lines and gives data regarding Fonda, Johnstown & Gloversville R. R.; one page.)

——Discussion. *Electric Railway Journal,* v. 38, p. 1210 (Dec. 9, 1911).

Glasgow, Scotland.

DEPRECIATION. (Editorial.) *Electrician,* v. 61, p. 744 (Aug. 28, 1908). (Practice of Glasgow Corporation Tramways.)

GLASGOW AND DEPRECIATION. *Municipal Journal* (London), v. 12, p. 795 (Sept. 4, 1903). (Very brief itemized statement of allowance for depreciation by Glasgow Corporation Tramways.)

GLASGOW TRAMWAYS. *Municipal Journal* (London), v. 14, p. 896 (Aug. 11, 1905). (Analysis of accounts of Glasgow Tramways, including allowance for depreciation.)

TREATMENT OF DEPRECIATION IN GLASGOW. *Electric Railway Journal,* v. 36, p. 362 (Sept. 3, 1910). (Discussion of the treatment of depreciation in accounts in report of the Glasgow Corporation Tramways.)

VALUATION OF GLASGOW TRAMWAYS. *Tramway and Railway World,* v. 27, p. 353 (May 5, 1910). (Decision in the appeal of the Glasgow Corporation Tramways against compulsory valuation.)

Great Britain.

B. E. T. DEPRECIATION. *Municipal Journal* (London), v. 16, p. 449 (May 24, 1907); v. 17, p. 459 (June 5, 1908). (Per cent. allowed for depreciation by sixteen British street railway companies.)

DEPRECIATION AND PERMANENT RENEWAL FUND; by William R. Bowker. *Street Railway Bulletin,* v. 6, p. 298 (May, 1907). (On the amount to be allowed for depreciation in street railroad property, giving cities of Manchester, Glasgow, Leeds, Bolton, and Wolverhampton, as examples.)

POLICY OF ENGLISH MUNICIPAL TRAMWAYS RESPECTING RENEWALS. *Electric Railway Journal,* v. 38, p. 661 (Oct. 7, 1911). (Two pages.)

RULES ON DEPRECIATION IN GREAT BRITAIN. *Electric Railway Journal,* v. 34, p. 476 (Sept. 25, 1909). (Allowances for depreciation in electric railway undertakings; one page.)

Illinois.

REPORT OF THE ILLINOIS TRACTION SYSTEM. (Editorial.) *Electric Railway Journal,* v. 36, p. 353 (Sept. 3, 1910). (Refers to annual allowance for depreciation.)

Kansas City, Mo.

DEPRECIATION CHARGES IN KANSAS CITY. (Editorial.) *Electric Railway Journal,* v. 36, p. 424 (Sept. 17, 1910). (On provision for depreciation made by the Kansas City Ry. & Light Co.; one paragraph.)

REPORT ON STREET RAILWAY SYSTEM OF KANSAS CITY. *Electric Railway Journal,* v. 41, p. 716 (April 19, 1913). (An investigation of the valuation of the property and its apportionment between the different municipalities.)

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

REPORT ON THE VALUE OF THE PROPERTIES OF THE METROPOLITAN STREET Railway System of Kansas City, v. 1; by Bion J. Arnold. Kansas City, Mo., 1913. (Report of Commission to investigate the capital value of the properties, various elements of such value, and how it shall be apportioned between the municipalities in a contract for a new franchise.)

A STREET-RAILWAY VALUATION. *Engineering News*, v. 69, p. 1053 (May 22, 1913). (Valuation of the street railway lines of Kansas City, Mo., by Bion J. Arnold; values found and various elements thereof and comparison with the values computed under four different methods.)

Kokomo, Marion & Western Traction Co.

DEPRECIATION ACCOUNT OF THE KOKOMO, MARION & WESTERN TRACTION Company. *Electric Railway Journal*, v. 38, p. 156 (July 22, 1911). (An arbitrary charge of a certain per cent. against each class or division of the property was adopted, to provide for current replacements and future requirements on account of losses due to age and wear; about the same percentages as are used by the Wisconsin Railroad Commission; one page.)

Lincoln, Nebr.

*TESTIMONY ON DEPRECIATION BEFORE NEBRASKA COMMISSION; by Edward W. Bemis. *Electric Railway Journal*, v. 35, p. 441 (March 12, 1910). (Testimony in relation to the consolidation of the properties comprising the Lincoln Traction Co.)

London, England.

LONDON TRAMWAY DEPRECIATION ALLOWANCE FOR INCOME TAX. *Electric Railway Journal*, v. 35, p. 274 (Feb. 12, 1910). (Very brief statement.)

Middlesex & Boston Street Ry.

LIABILITIES ON WHICH PROPER RETURNS SHOULD BE ALLOWED. *Electric Railway Journal*, v. 34, p. 464 (Sept. 25, 1909). (Hearing before Massachusetts Board of Railroad Commissioners; discussion of the basis on which the value of the property of the Middlesex & Boston Street Ry. should be computed.)

Milwaukee Electric Ry. & Light Co.

*DECISION IN THE MILWAUKEE FARE CASE. *Electric Railway Journal*, v. 40, p. 314 (Aug. 31, 1912). (Gives summary of physical valuation, going value, treatment of allowance for depreciation, rate of return, etc., of Milwaukee Light, Heat & Traction Co. and Milwaukee Electric Ry. & Light Co.)

DEPRECIATION AND RESERVE FUNDS IN MILWAUKEE. (Editorial.) *Street Railway Journal*, v. 26, p. 441 (Sept. 23, 1905). (Gives actual figures.)

DEPRECIATION OF PUBLIC UTILITIES PROPERTIES. *Electric Railway Journal*, v. 31, p. 169 (Feb. 1, 1908). (On the subject of depreciation in general, with reference to Milwaukee street railways.)

——Editorial. Depreciation. *Electric Railway Journal*, v. 31, p. 104 (Jan. 25, 1908).

DEPRECIATION RESERVES OF THE MILWAUKEE AND ST. LOUIS RAILWAYS. *Electric Railway Review*, v. 17, p. 319 (March 9, 1907). (Comparison of totals of maintenance and depreciation charges for fiscal year 1906, for street railways in Milwaukee, St. Louis, Chicago, and Glasgow.)

A DISCUSSION OF THE MILWAUKEE FARE DECISION. *Electric Railway Journal*, v. 41, p. 110 (Jan. 18, 1913). (Discusses valuation of Milwaukee street railway.)

THE ELECTRIC RAILWAY SYSTEM OF MILWAUKEE AND EASTERN WISCONSIN. *Street Railway Journal*, v. 15, p. 352 (June, 1899). (Provision of the Milwaukee Electric Ry. & Light Co., for depreciation and other reserves.)

——Editorial. *Street Railway Journal*, v. 15, p. 369 (June, 1899).

*HEARINGS ON MILWAUKEE FARE CASE BY WISCONSIN RAILROAD COMmission. *Electric Railway Journal*, v. 32, p. 395; v. 33, pp. 419, 464, 499, 554, 640, 683, 729, 766, 955 (Aug. 1, 1908; March 6, 13, 20, 27; April 3, 10, 17, 24; May 22, 1909). (Testimony by many experts on the value of the Milwaukee Electric Ry. & Light Co.'s property, allowances for depreciation, etc.)

——Editorial. *Electric Railway Journal*, v. 33, pp. 452, 536 (March 13, 27, 1909).

THE MILWAUKEE FOUR-CENT FARE DECISION. *Street Railway Journal*, v. 14, p. 397 (July, 1898). (Opinions of William H. Seaman, United States District Judge, in the case of the Milwaukee Electric Ry. & Light Co.; three pages.)

——Editorial. *Street Railway Journal*, v. 14, p. 381 (July, 1898).

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

VALUATION OF MILWAUKEE PROPERTIES. *Electric Railway Journal,* v. 38, p. 160 (July 22, 1911). (Details of the values placed on the property of the Milwaukee Electric Ry. & Light Co. by the Railroad Commission of Wisconsin; very brief.)

——Editorial. *Electric Railway Journal,* v. 38, p. 143 (July 22, 1911).

Milwaukee Northern Ry. Co.

*EDWARD J. CHROMASTER VS. MILWAUKEE NORTHERN RAILWAY COMPANY; Submitted May 15, 1911, Decided March 12, 1912. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 8, p. 734. Madison, Wis., 1912. (Table of total valuation of property of Milwaukee Northern Ry. Co. and apportionment between city and interurban.)

Nebraska.

*PROPOSED DEPRECIATION ACCOUNT IN NEBRASKA. *Electric Railway Journal,* v. 37, p. 919 (May 27, 1911). (Brief reference to hearing before the State Railway Commission on proposed depreciation account for electric railways.)

*RULE FOR TREATMENT OF DEPRECIATION IN NEBRASKA. *Electric Railway Journal,* v. 38, p. 990 (Nov. 4, 1911). (Rules adopted by State Railway Commission to govern charges by electric railways, for maintenance, additions and betterments; one-half page.)

New Jersey.

REQUEST FOR LOWER FARES DENIED BY NEW JERSEY COMMISSION. *Electric Railway Journal,* v. 38, p. 1117 (Nov. 25, 1911). (Contains brief reference to the value of street railway property and allowance for maintenance and depreciation.)

STANDARD CLASSIFICATION OF STREET RAILWAY ACCOUNTS IN NEW JERSEY. *Electric Railway Review,* v. 37, p. 273 (Feb. 11, 1911). (The adoption, with two slight changes, of the standards of the American Electric Railway Accountants Association in regard to depreciation.)

New York City.

ANOTHER THIRD AVENUE CHAPTER. (Editorial.) *Electric Railway Journal,* v. 39, p. 230 (Feb. 10, 1912). (The Commission provides a plan for the retirement of excessive capitalization of the property; very brief.)

*APPRAISAL OF THE NEW YORK SURFACE SYSTEMS. (Editorial.) *Electric Railway Journal,* v. 32, p. 111 (June 20, 1908). (Discussion of the proposed appraisal by the Public Service Commission; one page.)

THE APPRAISAL OF THE THIRD AVENUE STREET RAILROAD SYSTEM, NEW York City. *Engineering and Contracting,* v. 35, p. 666 (June 7, 1911). (Data relating to the appraisal taken from a pamphlet entitled "Opinion Disapproving Plan of Reorganization," July 29, 1910.)

AN ARBITRARY DEPRECIATION REQUIREMENT. (Editorial.) *Railway World,* v. 56, p. 141 (Feb. 16, 1912). (On decision of the New York Public Service Commission on the Third Avenue Ry.)

*LIFE OF ELEMENTS OF SUBWAY PROPERTY. *Electric Railway Journal,* v. 39, p. 575 (April 6, 1912). (Estimates of E. G. Connette, Transportation Engineer, New York Public Service Commission, First District.)

METROPOLITAN STREET RAILWAY REORGANIZATION. *Electric Railway Journal,* v. 37, pp. 708, 756, 798, 876, 916, 976; v. 38, p. 240 (April 22, 29, May 6, 20, 27, June 3, Aug. 5, 1911). (Hearing before the New York Public Service Commission, First District; testimony in relation to the value of the property.)

*OPINIONS OF COMMISSION IN THIRD AVENUE CASE. *Electric Railway Journal,* v. 39, p. 237 (Feb. 10, 1912). (Opinions on mortgages and accounting, amortization of discounts and depreciation of the Third Avenue R. R. property.)

PHYSICAL APPRAISAL OF THIRD AVENUE RAILROAD. *Electric Railway Journal,* v. 35, p. 228 (Feb. 5, 1910). (Estimate by Henry Floy of the value of the Third Avenue R. R.; one-half page.)

PLAN FOR REORGANIZATION OF THIRD AVENUE ROAD DISAPPROVED BY Commission. *Electric Railway Journal,* v. 36, p. 262 (Aug. 13, 1910). (Contains statement regarding the value of the property; three pages.)

*RE AMORTIZATION ACCOUNTS OF THE THIRD AVENUE RAILWAY COMPANY (Case 1181). Reports of Decisions of the Public Service Commission, First District of the State of New York, v. 3, p. 51. New York, 1912. (Amortization of discounts and depreciation.)

*RE BOND ISSUE OF NEW YORK & NORTH SHORE TRACTION COMPANY (CASE 1398). Reports of Decisions of the Public Service Commission, First District of the State of New York, v. 3, p. 63. New York, 1912. (Discussion of valuation of property of the New York & North Shore Traction Co.)

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

*RE METROPOLITAN STREET RAILWAY COMPANY REORGANIZATION (CASE 1305). Reports of Decisions of the Public Service Commission, First District of the State of New York, v. 3, p. 113. New York, 1912. (Estimates of valuation of property.)

*RE 2ND REORGANIZATION PLAN OF 3RD AVE. R. R. CO. (CASE 1181). Reports of Decisions of the Public Service Commission, First District of the State of New York, v. 2, p. 390. New York, 1912. (Report on valuation of the property of the Third Avenue R. R. Co.)

*REPORT OF A COMMITTEE OF THE BOARD OF ESTIMATE AND APPORTIONment and of the Public Service Commission for the First District with Relation to Pending Proposals for Rapid Transit Lines (1911). Proceedings, Public Service Commission for the First District, State of New York, v. 6, pp. 378, 399, 488 (June 27, 30, July 21, 1911). New York, 1912. (Estimated value of proposed New York Subway.)

*THE RETURN ON THE INVESTMENT IN THE SUBWAY OF THE INTERBOROUGH Rapid Transit Company of New York City, Submitted to the Public Service Commission for the First District of the State of New York, Report No. 7, Dec. 31, 1908; by Bion J. Arnold. New York, 1908. (Analysis of earnings and expenses, depreciation, etc.)

*SECOND CONDENSATION OF OPERATING ACCOUNTS FOR NEW YORK ROADS. *Electric Railway Journal,* v. 33, p. 67 (Jan. 9, 1909). (Classification of accounts made by New York Public Service Commission, First District; includes depreciation and treatment of appreciation.)

TENTATIVE CLASSIFICATION OF ACCOUNTS PREPARED BY NEW YORK PUBLIC Service Commission. *Electric Railway Journal,* v. 32, p. 349 (July 25, 1908). (The classification provides for two accounts to cover depreciation, one under maintenance of way and structures and one under maintenance of equipment.)

TREATMENT OF DEPRECIATION ACCOUNTS BY INTERBOROUGH RAPID TRANSIT Co. *Electric Railway Journal,* v. 38, p. 280 (Aug. 12, 1911). (One page.)

*TREATMENT OF DEPRECIATION AND MAINTENANCE IN GREATER NEW YORK. *Electric Railway Journal,* v. 39, p. 539 (April 6, 1912). (Table of rates of depreciation adopted by street and electric railway companies in accordance with the uniform system of accounts prescribed by the Public Service Commission of the First District.)

VALUATION OF STREET RAILWAY PROPERTIES. *Electric Railway Journal,* v. 33, p. 1122 (June 19, 1909). (Relates more particularly to the street railways of New York City; two and one-half pages.)

New York State.

*ACCOUNTS PRESCRIBED BY NEW YORK PUBLIC SERVICE COMMISSION, Second District. *Electric Railway Journal,* v. 32, p. 1373 (Nov. 14, 1908). (Provision is made for the treatment of depreciation in two primary operating expense accounts.)

BRIEF ON ACCOUNTING SCHEME SUBMITTED TO PUBLIC SERVICE COMMISSION, Second District, on Behalf of New York State Association. *Electric Railway Review,* v. 19, p. 591 (May 16, 1908). (Protest against requiring same methods of accounting for steam and electric railways and reasons for the protest).

INQUIRY BY PUBLIC SERVICE COMMISSION CONCERNING DEPRECIATION Accounts. *Electric Railway Journal,* v. 35, p. 793 (April 30, 1910). (Circular letter of inquiry issued to street railroad and electrical corporations, by the New York Public Service Commission, Second District; very brief.)

*JOINT HEARING ON UNIFORM ACCOUNTS FOR NEW YORK ELECTRIC ROADS. *Electric Railway Journal,* v. 32, p. 439 (Aug. 8, 1908). (Statement of H. J. Pierce, President of the International Ry. Co. of Buffalo, on depreciation, and of Howard Abel, Comptroller of the Brooklyn Rapid Transit System, on classification of accounts.)

RESOLUTIONS OF NEW YORK STATE ASSOCIATION CONCERNING TENTATIVE Classifications. *Electric Railway Review,* v. 19, p. 378 (March 28, 1908). (Relates to classification of the New York Public Service Commission, Second District.)

*SECOND CONDENSATION OF OPERATING EXPENSE ACCOUNTS FOR NEW YORK Roads. *Electric Railway Journal,* v. 33, p. 67 (Jan. 9, 1909). (Scheme of accounts prescribed by New York Public Service Commission, Second District, for street railroads; three paragraphs relating to depreciation.)

*STATE OF NEW YORK, SECOND ANNUAL REPORT OF THE PUBLIC SERVICE Commission, Second District, for the Year Ending Dec. 31, 1908; v. 2, Uniform System of Accounts. Albany, 1909. (Classification of accounts for street railroads, gas and electrical corporations; general amortization account including amount estimated for wear, tear and obsolescence of plant.)

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

***VALUATION BY EARNINGS**; by Frank W. Stevens. *Public Service Regulation*, v. 1, p. 438 (July, 1912). (Opinion of Chairman, New York Public Service Commission, Second District, in the application of the Westchester Street R. R. Co. for authorization to issue capital stock.)

VALUE OF PROPERTY IN NEW YORK REORGANIZATION CASE. *Electric Railway Journal*, v. 41, p. 381 (March 1, 1913). (Decision of the New York Public Service Commission, Second District, in the case in which the Westchester Street R. R. asked authority to issue capital stock.)

Newcastle, England.

DEPRECIATION AND RESERVES, WARNING TO NEWCASTLE. *Tramway and Railway World*, v. 20, p. 258 (Sept. 6, 1906). (Discussion on the necessity of reserve and renewal funds; statistical.)

Niagara Gorge R. R.

TREATMENT OF DEPRECIATION ACCOUNTS OF NEW YORK PUBLIC SERVICE Commission. *Electric Railway Journal*, v. 34, p. 1073 (Nov. 20, 1909). (Rules adopted by the Niagara Gorge R. R.)

Philadelphia, Pa.

***PENNSYLVANIA STATE RAILROAD COMMISSION IN THE MATTER OF THE** Complaints Against the Philadelphia Rapid Transit Company; Report to the Commission by Ford, Bacon & Davis, March 7, 1911. 2 v. New York, 1911. (Detailed report, comprising a series of tabulated statements, maps and diagrams on the physical valuation of the property of the Philadelphia Rapid Transit Co.)

Puget Sound Electric Ry.

***FIFTH ANNUAL REPORT OF THE RAILROAD COMMISSIONER OF WASHINGTON** to the Governor Covering the Period from January 1 to November 1, 1910, p. 49. Olympia, Wash., 1910. (Refers to valuation of the Puget Sound Electric Ry.)

VALUATION OF THE PUGET SOUND ELECTRIC RAILWAY; by Henry L. Gray. *Engineering-Contracting*, v. 33, p. 482 (May 25, 1910). (Methods and details of valuation of physical property; four pages.)

St. Louis, Mo.

DEPRECIATION FUND IN ST. LOUIS. (Editorial.) *Electric Railway Journal*, v. 35, p. 433 (March 12, 1910). (One paragraph.)

LARGER DEPRECIATION FUND FOR ST. LOUIS. (Editorial.) *Electric Railway Journal*, v. 37, p. 247 (Feb. 11, 1907). (Comments on the policy of the United Railways Co. of St. Louis.)

***REPORT TO THE MUNICIPAL ASSEMBLY ON THE UNITED RAILWAYS COM-**pany of St. Louis by the St. Louis Public Service Commission; by James E. Allison. 2 v. Woodward & Tiernan Printing Co., St. Louis, 1912. (A statement of the principles which in the opinion of the Commission should be the basis of valuation and details of physical valuation of the property; Appendix A contains a discussion by James E. Allison, "Should Public Service Properties be Depreciated to Obtain Fair Value in Rate or Regulation Cases?")

——Abstracts. Report on United Railways of St. Louis. *Electric Railway Journal*, v. 41, p. 248 (Feb. 8, 1913); Finding Fair Value. *Public Service Regulation*, v. 1, p. 716 (Nov., 1912).

San Francisco, Cal.

FINAL REPORT ON SAN FRANCISCO. *Electric Railway Journal*, v. 41, p. 844 (May 10, 1913). (Analysis of value of street railways of San Francisco; report by Bion J. Arnold.)

Savannah, Ga.

DECISION OF COMMISSION UPHOLDING RATES OF FARE IN SAVANAH, GA. *Electric Railway Journal*, v. 39, p. 663 (April 20, 1912). (Relates to value of street railway property.)

Sheboygan, Wis.

***CITY OF SHEBOYGAN VS. SHEBOYGAN RAILWAY AND ELECTRIC COMPANY;** Submitted Oct. 18, 1910, Decided Feb. 3, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 6, p. 358. Madison, Wis., 1912. (Contains table of valuation of the physical property of the Sheboygan Ry. & Electric Co.)

Spokane & Inland Empire R. R. System.

APPRAISAL OF THE SPOKANE AND INLAND EMPIRE ELECTRIC RAILROAD System; by Henry L. Gray. *Engineering and Contracting*, v. 36, p. 696 (Dec. 27, 1911). (Contains tables of cost of reproduction, depreciation, etc.; deals with methods adopted to determine the correctness of the allegation concerning the insufficiency of present rates.)

STREET AND INTERURBAN RAILROADS—SPECIAL CASES—(*Continued*).

Sunderland, England.

DEPRECIATION AT SUNDERLAND. *Tramway and Railway World,* v. 14, p. 478 (Nov. 12, 1903). (Estimated life and percentage set aside for depreciation of equipment and permanent way of street railway at Sunderland, England.)

Toledo, Ohio.

INVENTORY OF PHYSICAL RAILWAY PROPERTY OF TOLEDO COMPANY GIVEN to City. *Electric Railway Journal,* v. 36, p. 990 (Nov. 12, 1910). (Letter from Ford, Bacon & Davis; gives a list of the work and expense items entering into the construction cost which should be included in an appraisal; one page.)

Wausau, Wis.

***DECISION OF WISCONSIN COMMISSION: CONSIDERS THE RATE OF RETURN.** *Electric Railway Journal,* v. 36, p. 404 (Sept. 10, 1910). (Statement in regard to the valuation of the Wausau Street R. R. Co.; one page.)

Wisconsin.

SOME PRINCIPLES ESTABLISHED BY THE WISCONSIN RAILROAD COMMISSION; by Edwin S. Mack. *Electric Railway Journal,* v. 37, p. 164 (Jan. 28, 1911). (Includes valuation; one page.)

***SYSTEM OF ACCOUNTS PRESCRIBED BY WISCONSIN RAILROAD COMMISSION.** *Electric Railway Journal,* v. 33, p. 1076 (June 12, 1909). (Includes provision for a depreciation fund; two pages.)

***WISCONSIN CLASSIFICATION OF ACCOUNTS.** *Electric Railway Journal,* v. 34, p. 366 (Sept. 4, 1909). (Gives slight changes from list published June 12, 1909, with extracts from introductory letter of the Commission.)

STREET AND INTERURBAN RAILROADS—UNVERIFIED REFERENCES.

ACCOUNTING OF DEPRECIATION BY ELECTRIC RAILWAYS; by Robert N. Wallis. *Journal of Accountancy,* May, 1907. (Discusses methods used and matters to be considered in electric railway accounting.)

ALLOWANCE FOR MAINTENANCE TO COVER DEPRECIATION. *Light Railway and Tramway Journal* (London), v. 21, p. 29 (July 2, 1909). (Details of allowances as agreed between Council of Institute of Municipal Treasurers and Accountants and the Inland Revenue Department for tramway and light railways as well as for municipal corporations.)

CLASSIFICATION OF ELECTRIC RAILWAY EXPENSES; by Willard Hubbard Lawton. *Journal of Accountancy,* v. 6, p. 114 (June, 1908).

COST OF CARRYING A PASSENGER; by C. L. S. Tingley. *Street Railway Bulletin,* v. 4, p. 793 (Nov. 15, 1905). (Abstract.)

DEPRECIATION AND RENEWAL FUNDS IN RELATION TO TRAMWAYS UNDERtakings; by G. W. Holford. *Light Railway and Tramway Journal* (London), v. 15, p. 294 (Oct. 5, 1906). (Abstract of paper read before the Municipal Tramways Association; contains table showing provision made for depreciation by street railways in sixty English cities.)

DEPRECIATION OF TRAMWAYS AND LIGHT RAILWAYS. *Light Railway and Tramway Journal* (London), v. 21, p. 80 (Aug. 6, 1909).

DEPRECIATION PROBLEM OF ELECTRIC STREET RAILWAYS; by W. B. Jackson. *Public Service,* Sept., 1911, p. 71.

LIFE OF DIFFERENT PARTS OF CAR EQUIPMENT, BEFORE AND AFTER USING Recording Wattmeters on Cars at Cape Town. *Journal,* Tramways and Light Railway Association, Sept., 1911, p. 241.

MANAGEMENT OF ELECTRIC TRAMWAYS; by Bowker.

PHYSICAL VALUE SCHEDULES OF THE CLEVELAND ELECTRIC RAILWAY CO. as of January 1, 1908.

REPORT OF THE STREET RAILWAY COMMISSION TO THE DETROIT COMMON Council on the Valuation of the Street Railways of Detroit. Journal of the Common Council, City of Detroit, 1899, p. 346.

REPORTING A STREET RAILWAY EXAMINATION (FROM THE CLIENT'S POINT of View); by W. B. Brockway. *Journal of Accountancy,* v. 4, p. 16 (May, 1907).

STREET RAILWAY COSTS; by M. E. Cooley. *Public Service,* v. 6, p. 117 (April, 1909).

STREET RAILWAY FARES IN LARGE CITIES; by Howard S. Knowlton. *Review of Reviews,* v. 32, p. 80 (July, 1905).

STREET RAILWAY SETTLEMENT IN CLEVELAND; by E. W. Bemis. *Quarterly Journal of Economics,* Aug., 1908. (Thirty-three pages.)

STREET AND INTERURBAN RAILROADS—UNVERIFIED REFERENCES—(*Continued*)

TRAMWAY BOOKKEEPING ACCOUNTS; by D. McGall.

VALUATION OF CLEVELAND RAILWAY CO.: LETTER OF DEC. 17, 1909, TO CITY Council of Cleveland, Ohio, by R. W. Taylor, as Arbitrator Between City and Railways. (Copy in files, American Electric Railway Association.)

VALUATION OF PROPERTY OF RAILROADS IN THE DISTRICT OF COLUMBIA. 62d Cong., 2d Sess. Senate Doc. No. 835. (Letter in response to Senate resolution of Feb. 14th, 1912.)

TELEGRAPH AND TELEPHONE.

AN ANALYSIS OF COST OF TELEPHONE SERVICE. *Electrical World,* v. 55, p. 1243 (May 19, 1910). (One page.)

DEPRECIATION AND REPLACEMENT OF GROWING TELEPHONE PLANTS; by Burke Smith. *Journal,* Western Society of Engineers, v. 17, p. 779 (Oct., 1912). (Discusses renewal, life and depreciation of plant.)

——Abstract. Depreciation and Replacement of Telephone Equipment. *Electrical Review and Western Electrician,* v. 60, p. 790 (April 27, 1912). (Very brief.)

THE DEPRECIATION OF UNDERGROUND CABLES; by F. Fernie. *Electrical Review* (London), v. 60, p. 577 (April 5, 1907). (Method of computing depreciation of cables.)

DEPRECIATION OF UNDERGROUND CONDUITS, CABLES AND WIRES. *Electric Railway Journal,* v. 33, p. 881 (May 8, 1909). (Classification of property of the American Telephone & Telegraph Co.; very brief.)

DETERMINATION OF TELEPHONE RATES FOR LARGE EXCHANGES; by William H. Crumb. *Journal,* Western Society of Engineers, v. 12, p. 781 (Dec., 1907). (On depreciation, with a table giving per cent. to be allowed to depreciation account; one-half page.)

PLANT-INVENTORY AND VALUATION. *Electrical World,* v. 55, p. 295 (Feb. 3, 1910). (Abstract of paper by W. R. McGovern before the Wisconsin Electrical Association covering in a general way methods of taking inventories of telephone-exchange plants in Wisconsin.

——Abstract and Discussion. *Electric Railway Journal,* v. 35, p. 184 (Jan. 29, 1910).

TELEPHONE CONSTRUCTION METHODS AND COST. Chicago, 1908. (Contains cost data from the actual records of various telephone companies.)

*UNIFORM SYSTEM OF ACCOUNTS FOR TELEPHONE COMPANIES AS PRE-scribed by the Interstate Commerce Commission, pp. 16, 34, 53, 66, 71, 77. Washington, 1912. (Refers to treatment of depreciation.)

TELEGRAPH AND TELEPHONE—SPECIAL CASES.

Augusta, Wis.

*IN RE APPLICATION OF J. L. BALL FOR AUTHORITY TO INCREASE TELE-phone Rates; Submitted Sept. 10, 1907, Decided Nov. 25, 1907. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 2, p. 105. Madison, Wis., 1909. (Brief reference to physical valuation.)

Boston, Mass.

REPORT TO THE MASSACHUSETTS HIGHWAY COMMISSION ON TELEPHONE Rates for Boston and Suburban District; by D. C. and William B. Jackson. Boston, 1910. (Diagrams, tables and maps; sixty-six pages.)

Chicago, Ill.

THE APPRAISAL OF THE CHICAGO TELEPHONE CO. AND A COMPARISON WITH the Results of Three Similar Appraisals. (Editorial.) *Engineering and Contracting,* v. 36, p. 269 (Sept. 13, 1911). (One column.)

APPRAISAL OF THE CHICAGO TELEPHONE COMPANY AND DETERMINATION of Fair Rates of Charge; by William J. Hagenah. *Engineering-Contracting,* v. 36, pp. 289, 445, 473 (Sept. 13, Oct. 25, Nov. 1, 1911). (Method of arriving at plant value, appraised value and income and operating expenses.)

REPORT ON THE TELEPHONE SITUATION IN THE CITY OF CHICAGO IN RESPECT to Service, Rates, Regulation of Rates, etc., Submitted to the Committee on Gas, Oil and Electric Light of the City Council of the City of Chicago; by a Special Committee, composed of Dugald C. Jackson, William H. Crum, and George W. Wilder, April, 1907. Chicago, 1907. (The Committee states that it has endeavored to obtain data from Bell Telephone Companies of New York and elsewhere, but rates seem to have been dictated by estimates based on experience or the requirements of business expediency.)

TELEGRAPH AND TELEPHONE—SPECIAL CASES—(*Continued*).

Clinton Telephone Co.

***B. B. TIGHE ET AL. VS. CLINTON TELEPHONE COMPANY;** Submitted Oct. 10, 1908, Decided Dec. 2, 1908. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 3, p. 117. Madison, Wis., 1910. (Refers to tentative valuation of physical property.)

Massachusetts.

SIXTEENTH ANNUAL REPORT OF THE MASSACHUSETTS HIGHWAY COMMISsion, for the Fiscal Year ending Nov. 30, 1908, p. 158. Boston, 1909. (Contains report by Dugald C. Jackson on the advisability of making an appraisal of the New England Telephone & Telegraph Co.)

——17th, p. 211. Boston, 1910. (Contains a summary report of the results of the inventory and appraisal of the New England Telephone & Telegraph Co., by D. C. and W. B. Jackson.)

TELEPHONE REPORT TO MASSACHUSETTS HIGHWAY COMMISSION. *Electrical World,* v. 55, p. 984 (April 21, 1910). (Brief reference to report on the New England Telephone & Telegraph Co., by D. C. and W. B. Jackson; states that this report on traffic and operating conditions, together with the appraisal which appeared previously, is the most detailed analysis ever made of telephone rates in an urban area of such scope.)

Michigan.

THE VALUATION AND TAXATION OF TELEPHONE COMPANIES IN MICHIGAN; by W. J. Rice. *Electrical World,* v. 37, p. 196 (Feb. 2, 1901). (Two and one-half pages.)

Portage, Wis.

***IN RE APPLICATION OF THE PORTAGE TELEPHONE COMPANY FOR AUTHOR**ity to Increase Rates; Submitted May 19, 1908, Decided Aug. 27, 1908. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 2, p. 692. Madison, Wis., 1909. (Contains brief data on valuation.)

Seattle, Wash.

APPRAISAL OF THE PACIFIC TELEPHONE AND TELEGRAPH COMPANY OF Seattle; by Henry L. Gray. *Engineering and Contracting,* v. 36, p. 332 (Sept. 27, 1911). (Gives detailed estimate of cost of reproducing the plant; five pages.)

APPRAISAL OF THE SEATTLE TELEPHONE COMPANIES BY THE RAILROAD Commission of Washington; by Henry L. Gray. *Engineering and Contracting,* v. 35, p. 520 (May 3, 1911). (Describes the work and explains the causes which led to the appraisal.)

STUDY OF THE TELEPHONE SITUATION IN SEATTLE, WASH.: REPORT; by C. H. Judson and F. B. Hall. *Engineering News,* v. 65, p. 652 (June 1, 1911). (On depreciation and valuation; one column.)

Wisconsin Telephone Co.

***E. E. PAYNE ET AL. VS. WISCONSIN TELEPHONE COMPANY;** Decided Aug. 3, 1909. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 4, p. 1. Madison, Wis., 1910. (Intangible values embracing franchise, good will and going value are briefly discussed.)

TELEGRAPH AND TELEPHONE—UNVERIFIED REFERENCES.

DEPRECIATION AND REPLACEMENT OF GROWING TELEPHONE PLANTS; by Burke Smith. *Telephony,* April 27, 1912, p. 1062.

NATIONAL TELEPHONE CO., LIMITED, VS. HIS MAJESTY'S POSTMASTER-General. Great Britain Railway and Canal Commission Court. Judgment, Jan. 13, 1913, pp. 4209-39.

REPORT OF THE INVESTIGATION OF THE CHICAGO TELEPHONE COMPANY, 1911; by William J. Hagenah.

REPORT OF UNFAIR TELEPHONE RATES IN MINNEAPOLIS, TAKES UP ALLOWance for Depreciation; by Gordon Steele & Co. *Public Service,* v. 5, p. 106 (Oct., 1908).

TELEPHONE VALUATION, OKLAHOMA. Pioneer Telephone & Telegraph Co. *vs.* E. S. Westenhaver *et al.* and State of Oklahoma. Oklahoma Supreme Court, 1910, p. 1256.

WATER POWER—GENERAL.

AN ANALYSIS OF THE "COMMERCIAL" VALUE OF WATER-POWER PER Horse-power per Annum; by A. F. Nagle. *Transactions,* American Society of Mechanical Engineers, v. 24, p. 286 (1903). (With discussion.)

——Abstract. *Engineering News,* v. 49, p. 83 (Jan. 22, 1903).

WATER POWER—GENERAL—(*Continued*).

THE APPRAISAL OF WATER POWER RIGHTS; by Halbert P. Gillette. *Engineering and Contracting*, v. 38, p. 624 (Dec. 4, 1912). (Discusses seven methods of appraisal: the steam comparison method; the capitalized profit method; the alternative use method; the next available water power method; the public utility factor method; the market value method; and capitalized rental method.)

COMMENTS ON DEVELOPMENT AND OPERATION OF HYDRO-ELECTRIC PLANTS; by Henry L. Doherty. *Proceedings*, American Institute of Electrical Engineers, v. 28, Pt. 2, p. 1362 (1909). (Treat of value of water-power securities, cost of power and depreciation.)

——Abstract. Cost and Depreciation of Steam and Hydro-Electric Installations. *Electrical World*, v. 54, p. 1558 (Dec. 30, 1909).

COMPARATIVE COST OF STEAM AND WATER POWER; by Charles H. Manning. *Transactions*, American Society of Mechanical Engineers, v. 10, p. 499 (1889). (On the value of water power.)

THE COMPARATIVE VALUES OF WATER-POWER AND STEAM POWER; by A. C. Dunham. *Electrical World*, v. 59, p. 38 (Jan. 6, 1912). (Advantages and disadvantages of hydro-electric stations for central-station service compared with steam-driven stations, financial considerations involved.)

——Abstract. The Value of Water Power. *Engineering Record*, v. 65, p. 181 (Feb. 17, 1912).

COMPUTATION OF THE VALUE OF WATER POWERS AND THE DAMAGES Caused by the Diversion of Water Used for Power; by Charles T. Main. *Journal*, New England Water Works Association, v. 21, p. 214 (Sept., 1907). (Contains a table of estimated costs per horse-power of water-power plants.)

——Abstracts. *Engineering-Contracting*, v. 29, p. 73 (Jan. 29, 1908); *Journal of Electricity, Power and Gas*, v. 22, p. 20 (June 30, 1909).

COMPUTATION OF THE VALUES OF WATER POWERS AND THE DAMAGES Caused by the Diversion of Water Used for Power; by Charles T. Main. *Transactions*, American Society of Mechanical Engineers, v. 26, p. 68 (1905). (Gives definitions of value and damage and method of determining value.)

COST OF GENERATING ELECTRIC POWER; by F. A. Griffin. *Street Railway Journal*, v. 26, p. 1142 (Dec. 30, 1905). (Gives diagram of maintenance of generating station and states that the cost of generating power by water can be found from the diagram by eliminating the cost of coal.)

COST OF STEAM AND WATER POWER; by Charles T. Main. *Transactions*, American Society of Mechanical Engineers, v. 11, p. 108 (1889). (On value of water power.)

THE COST OF STEAM POWER; by Charles E. Emery. *Transactions*, American Society of Civil Engineers, v. 12, p. 425 (Paper 266. Nov., 1883). (In a suit respecting the loss of power by the diversion of water, damages were claimed by mill owners based on the cost of purchasing, operating and maintaining at each mill a small engine and a complete independent steam plant.)

DAMAGES CAUSED BY THE DIVERSION OF WATER-POWER; by Clemens Herschel. *Journal*, New England Water Works Association, v. 21, p. 241 (Sept., 1907). (On methods of estimating market value of water power.)

ELECTRIC POWER TRANSMISSION; by Frederick Darlington. *Transactions*, American Institute of Electrical Engineers, v. 25, p. 181 (1906). (Cost of developing and maintaining hydro-electric power utilized in mill work, in comparison with steam power.)

ELECTRIC TRANSMISSION OF WATER-POWER, p. 1; by Alton D. Adams. McGraw Publishing Co., New York, 1906. (On water-power and its utility in electrical supply, relates to the cost of water power; two chapters.)

EVALUATION OF WATER RIGHTS; by Arthur Halsted. *Engineering Record*, v. 61, p. 245 (Feb. 26, 1910). (On methods of valuation; two pages.)

HYDRAULIC DEVELOPMENTS AS RELATED TO ELECTRIC INSTALLATIONS; by William B. Jackson. *Journal*, Western Society of Engineers, v. 8, p. 312 (June, 1903). (Contains a discussion by T. T. Johnson on the economic side of a water-driven electric plant.)

HYDRO-ELECTRIC POWER VERSUS STEAM FOR INDUSTRIAL PURPOSES; by H. von Schon. *Engineering Magazine*, v. 33, pp. 34, 184, 353 (April, May, June, 1907). (A comparison of costs of hydro-electric and steam power in developed and undeveloped plants.)

A METHOD OF ESTIMATING THE LOSS OF WATER POWER IN A STREAM BY Taking Water Therefrom for a City Supply; by L. M. Hastings. *Journal*, New England Water Works Association, v. 7, p. 187 (June, 1893). (Discussion on the value of the water power.)

——Abstract. *Engineering Record*, v. 27, p. 296 (March 11, 1893).

WATER POWER—GENERAL—(*Continued*).

NOTES ON DESIGN OF HYDRO-ELECTRIC POWER STATIONS, WITH REFERENCE to the Influence of Load-Factor; by David B. Rushmore. *Transactions,* American Institute of Electrical Engineers, v. 25, p. 145 (1906). (The discussion of costs is confined to general principles; definite values are not given.)

ON THE VALUE OF A HORSE-POWER; by George I. Rockwood. *Transactions,* American Society of Mechanical Engineers, v. 21, p. 590 (1900). (On the dispute between the city officials of Worcester and the mill owners on the loss of power by the diversion of water for the Worcester Reservoirs.)

THE RELATION OF LOAD-FACTOR TO THE EVALUATION OF HYDRO-ELECTRIC Plants; by S. B. Storer. *Transactions.* American Institute of Electrical Engineers, v. 25, p. 139 (1906). (Contains diagrams giving comparative cost of steam and hydro-electric plants.)

RELATION OF STEAM TO WATER POWER; by James G. Hill. *Transactions,* New England Cotton Manufacturers' Association, v. 65, p. 333 (1898). (Contains a table giving yearly cost of 1 h.p. of water.)

THE SALE AND MEASUREMENT OF ELECTRIC POWER; by S. B. Storer. *Street Railway Journal,* v. 27, p. 1018 (June 30, 1906). (Contains diagrams giving methods of plotting costs and prices per horse-power per year of steam or hydro-electric power.)

STORAGE AND PONDAGE OF WATER; by Joseph P. Frizell. *Transactions,* American Society of Civil Engineers, v. 31, pp. 29, 552 (Papers 688 and 709. Jan., May, 1894). (On the valuation of water power.)

VALUATION OF MANUFACTURING PROPERTY FOR TAXATION; by Charles T. Main. *Transactions,* New England Cotton Manufacturers Association, v. 67, p. 108 (1899). (Contains data on the method of determining the value of water power.)

THE VALUATION OF WATER POWER AND THE ESTIMATION OF STREAM Diversion Damages; by Robert E. Horton. *Proceedings,* American Water Works Association, v. 29, p. 1 (1909). (Twenty-five pages.)

THE VALUE AND DESIGN OF WATER POWER PLANTS AS INFLUENCED BY Load-Factor; by Frederick A. C. Perrine. *Journal,* Franklin Institute, v. 162, p. 269 (Oct., 1906).

THE VALUE OF A WATER POWER; by Charles T. Main. *Transactions,* American Society of Mechanical Engineers, v. 13, p. 140 (1891). (On method of estimating the value of water power with relation to condemnation proceedings.)

THE VALUE OF WATER; by Alton D. Adams. *Municipal Engineer,* v. 37, p. 77 (Aug., 1909). (Decision of the Massachusetts Court as to the general rule of compensation where water is taken under legislative authority for public use.)

——Editorial. Value of Water and Water Power. *Municipal Engineering,* v. 37, p. 102 (Aug., 1909).

THE VALUE OF WATER POWER. (Editorial.) *Engineering Record,* v. 33, p. 597 (May 9, 1896). (Dealing with the general and special conditions and circumstances that should be considered in the condemnation of water supplies for municipal purposes; two columns.)

WATER POWER. In Geological Survey of New Jersey, Final Report, v. 3, Henry B. Kummel, State Geologist. Trenton, N. J., 1894. (Data on the cost of water power.)

WATER RIGHTS; by Richard A. Hale. *Journal,* New England Water Works Association, v. 21, p. 248 (Sept., 1907). (The discussion of this paper relates to the valuation of water power.)

WATER POWER—SPECIAL CASES.

Beloit, Wis.

***CITY OF BELOIT VS. BELOIT WATER, GAS AND ELECTRIC COMPANY;** Decided July 19, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 246. Madison, Wis., 1912. (Gives estimates of value of the water power.)

***VALUATION OF WATER POWER.** *Engineering Record,* v. 61, p. 153 (Feb. 5, 1910). (Case of the Beloit Water, Gas & Electric Co. before the Wisconsin Railroad Commission; abstracts of testimony.)

Boston, Mass.

BOSTON BELTING COMPANY VS. CITY OF BOSTON. Boston, 1899. (Suit relates to the diversion of water from Stony Brook and the value of the water power; fifty-nine parts.)

WATER POWER—SPECIAL CASES—(*Continued*).

Bronx River.

CALCULATIONS OF THE MEAN HORSE-POWER OF A VARIABLE STREAM AND the Cost of Replacing the Power Lost by a Partial Diversion of the Flow; by William H. Grant. *Transactions*, American Society of Civil Engineers, v. 22, p. 389 (Paper 440. June, 1890). (In regard to the claims of property owners for diversion of part of the waters of the Bronx River.)

Connecticut River Transmission Co.

HEARING ON CONNECTICUT RIVER TRANSMISSION COMPANY'S FRANCHISE. *Electrical World*, v. 53, p. 486 (Feb. 25, 1909). (Brief data on the estimated value of the plant.)

Everett (Wash.) Ry., Light & Power Co.

*THE APPRAISAL OF WATER RIGHTS. *Engineering and Contracting*, v. 37, p. 429 (April 17, 1912). (Report made by Halbert P. Gillette to the Everett Ry., Light & Water Co. as part of the Company's exhibits in a recent hearing before the Washington Public Service Commission.)

Holyoke, Mass.

HOLYOKE WATER POWER CO. VS. CITY OF HOLYOKE. 20 v. George H. Ellis, Boston, 1899. (Suit in regard to the value of the water power.)

Illinois.

REPORT ON THE WATER POWER OF THE ROCK RIVER AT STERLING AND Rock Falls, Illinois, and the Probable Effect on the Power of the Diversion of Water for Feeding the Illinois and Mississippi Canal; by Daniel W. Mead. The Author, Chicago, 1904. (Gives the value of water power destroyed by diversion of water.)

Kaukauma, Wis.

*IN RE DETERMINING AND FIXING THE JUST COMPENSATION TO BE PAID TO the Kaukauma Gas, Electric Light & Power Company by the City of Kaukauma, for the Property of Said Company Actually Used and Useful for the Convenience of the Public; Submitted Feb. 6, 1911, Decided Dec. 26, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 8, p. 417. Madison, Wis., 1912. (On the value of a water-power lease.)

Kennebec River.

WATER-POWER OF CARATUNK FALLS, KENNEBEC RIVER, MAINE; by Samuel McElroy. *Transactions*, American Society of Mechanical Engineers, v. 17, p. 58 (1895-96). (On the relative cost of steam and water power.)

——Abstract. *Engineering News*, v. 34, p. 422 (Dec. 26, 1895).

Nashua River Paper Co.

NASHUA RIVER PAPER COMPANY VS. COMMONWEALTH OF MASSACHUSETTS. 4 v. Wright & Potter Printing Co., Boston, 1901. (On damages to be awarded for drawing off the water flowing by the plaintiff's mills.)

Newton, N. J.

AN IMPORTANT DECISION UPON A MILL OWNER'S SUIT TO COLLECT DAMages for the Diversion of Water. (Letter); by Louis L. Tribus. *Engineering News*, v. 43, p. 228 (April 5, 1900). (Gives text of decision in the case at Newton, N. J.)

NOTES ON NEWTON, N. J., WATER WORKS CONSTRUCTION AND LITIGATION; by Louis L. Tribus. *Journal*, New England Water Works Association, v. 23, p. 145 (June, 1909). (Decision of Court as to valuation of water power diverted for Newton, N. J., Water-Works.)

Niagara Falls.

COURT OF APPEALS: THE PEOPLE OF THE STATE OF NEW YORK, EX REL. Niagara Falls Hydraulic Power and Manufacturing Company, Appellant, Against State Board of Tax Commissioners, Respondents: Respondents' Points. Buffalo, 1911. (On valuation of property of the Niagara Falls Hydraulic Power Co.)

COURT OF APPEALS: THE PEOPLE OF THE STATE OF NEW YORK, EX REL. the Niagara Falls Hydraulic Power and Manufacturing Company, Appellant, *versus* the State Board of Tax Commissioners, Respondents, John L. Romer, Counsel for Appellant: Points for Appellant. Buffalo, 1910. (Relates to valuation of property of the Niagara Falls Hydraulic Power Co.)

SUPREME COURT, ERIE COUNTY: THE PEOPLE OF THE STATE OF NEW YORK, *ex rel.* the Niagara Falls Hydraulic Power & Manufacturing Company, against the State Board of Tax Commissioners. no place, no date. (Decision in regard to the valuation of property of the Niagara Falls Hydraulic Power Co.)

WATER POWER AT NIAGARA FALLS; by Samuel McElroy. *Journal*, Association of Engineering Societies, v. 4, p. 395 (Sept., 1885). (Estimation of the value of water power by Commissioners to award damages.)

WATER POWER—SPECIAL CASES—(*Continued*).

Troy, N. Y.

ABSTRACT FROM THE REPORT OF THE WATER COMMISSIONERS, TROY, N. Y., Damages to Mill Powers; by William G. Raymond. *Journal,* New England Water Works Association, v. 13, p. 152 (Dec., 1898). (On the method of estimating value of water power; cost of damages.)

Waltham, Mass.

WATER POWER, ITS MEASUREMENTS AND VALUE, WITH DATA RESPECTING Damages Awarded; by George A. Kimball. *Journal,* Association of Engineering Societies, v. 13, p. 71 (Feb., 1894). (Paper read before the Boston Society of Civil Engineers; some of the facts presented in evidence before the commission to award damages in the City of Waltham, Mass.)

Wausau, Wis.

***FRANK B. L. FULLMER VS. WAUSAU STREET RAILROAD COMPANY;** Decided April 1, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 125. Madison, Wis., 1911. (Gives basis of determining value of water power.)

Worcester, Mass.

KETTLE BROOK AND BLACKSTONE VALLEY MILL OWNERS, PETITIONERS, VS. City of Worcester. 5 v. Worcester, 1899. (Final arguments in a suit on the loss of power from the diversion of water.)

WATER-WORKS—GENERAL.

THE APPRAISAL AND DEPRECIATION OF WATER WORKS AND SIMILAR PROPerties; by William H. Bryan. *Journal,* Association of Engineering Societies, v. 39, p. 336 (Dec., 1907). (Forty-five pages.)

——Abstract. Depreciation Allowances for Various Public-Service Industries. *Engineering News,* v. 59, p. 96 (Jan. 23, 1908).

THE APPRAISAL OF PLANTS FOR PUBLIC SERVICES; by Nicholas S. Hill, Jr. *Engineering Record,* v. 43, p. 546 (June 8, 1901). (On methods of evaluation of water-works, etc., by the city.)

THE APPRAISAL OF WATER WORKS PROPERTIES WITH SPECIAL REFERENCE to the Reproduction Method. *Engineering and Contracting,* v. 39, p. 420 (April 9, 1913). (Paper read by Douglas A. Graham before the Illinois Water Supply Association.)

——*Engineering News,* v. 69, p. 677 (April 3, 1913).

——*Engineering Record,* v. 67, p. 532 (May 10, 1913).

THE ARBITRATION OR APPRAISAL OF THE VALUE OF PUBLIC UTILITIES; by Daniel W. Mead. *Proceedings,* American Water Works Association, v. 22, p. 132 (1902). (On the appointment of boards, duties of appraisers, evidence on the part of the company and of the municipality, and basis of valuation.)

CONSTRUING CERTAIN CLAUSES IN MUNICIPAL FRANCHISES; by John W. Hill. *Proceedings,* American Water Works Association, v. 19, p. 223 (1899). (On the value of water-works plants.)

——Abstract. *Engineering Record,* v. 39, p. 594 (May 27, 1899).

DEPRECIATION; by John W. Alvord. *Proceedings,* American Water Works Association, v. 23, p. 473 (1903). (Takes up the physical depreciation of pumping engines, stand-pipes, boilers, etc.; fifty-one pages.)

DEPRECIATION AS APPLIED TO WATER WORKS PROPERTIES; by Morris Knowles. no place, 1910. (Eighteen pages.)

DEPRECIATION CHARGES. (Letter); by Charles W. Helwick. *Engineering Record,* v. 59, p. 672 (May 29, 1909). (On valuation of water-works; very brief.)

DEPRECIATION IN CAST IRON PIPE. *Municipal Engineering,* v. 41, p. 211 (Sept., 1911). (Estimates for purpose of valuation.)

DEPRECIATION IN WATER WORKS ACCOUNTS, WITH REFERENCE TO UNIform Reports; by Harvey S. Chase. *Journal,* New England Water Works Association, v. 24, p. 305 (June, 1910). (Discusses depreciation in general as well as its relation to accounts; twenty-six pages.)

——Abstract. *Engineering-Contracting,* v. 33, p. 598 (June 29, 1910).

——Editorial. Depreciation and Sinking Fund Accounts. *Municipal Engineering,* v. 39, p. 26 (July, 1910).

DEPRECIATION IN WATER WORKS OPERATION AND ACCOUNTING; by Leonard Metcalf. *Journal,* New England Water Works Association, v. 24, p. 442 (Dec., 1910). (Thirty pages, with diagrams.)

WATER-WORKS—GENERAL—*(Continued)*.

——Abstracts. *Engineering News,* v. 64, p. 482 (Nov. 3, 1910); Methods of Computing Depreciation of and Reasonable Assumptions as to Period of Useful Life of Water Works. *Engineering and Contracting,* v. 34, p. 434 (Nov. 16, 1910).

——Discussion. Methods of Making Computations for Depreciation in Public Utility Plants. *Engineering-Contracting,* v. 34, p. 590 (Dec. 28, 1910).

A DISCUSSION OF DEPRECIATION AND A COMPARISON OF RATES AND BOOK-keeping Methods of Municipally and Privately Owned Water Works. *Proceedings,* American Water Works Association, v. 32, p. 325 (1912). (Discusses report of Committee on Depreciation; twenty-two pages.)

——Abstract. Depreciation; by John W. Alvord. *Engineering Record,* v. 65, p. 670 (June 15, 1912).

THE FINANCIAL MANAGEMENT OF WATER-WORKS; by E. Kuichling. *Transactions,* American Society of Civil Engineers, v. 38, p. 1 (Paper 809. Dec., 1897). (Discusses annual expenses of water-works, including depreciation fund, yearly extensions, minor betterments, and future additions.)

THE FINANCIAL MANAGEMENT OF WATER WORKS; by Freeman C. Coffin. *Journal,* New England Water Works Association, v. 11, p. 63 (Sept., 1896). (Gives tables of water-works statistics; original cost, cost of annual maintenance, etc.)

FINANCIAL QUESTIONS IN WATER-WORKS VALUATION; by John W. Alvord. *Proceedings,* American Water Works Association, v. 22, p. 142 (1902). (On the business value, the franchise value and the fair return.)

——Abstracts. *Engineering Record,* v. 46, p. 30 (July 12, 1902); *Engineering News,* v. 47, p. 509 (June 19, 1902).

THE GOING VALUE OF WATER WORKS; by Leonard Metcalf and John W. Alvord. *Transactions,* American Society of Civil Engineers, v. 73, p. 334 (Paper 1199. Sept., 1911). (Sixty-six pages.)

METHOD OF ARBITRATION WITH REFERENCE TO VALUE OF WATER WORKS Plants; by Peter Milne. *Proceedings,* American Water Works Association, v. 17, p. 25 (1897). (On the tangible property and its value, value of franchise, and their relation; two and one-half pages.)

MUNICIPAL WATER SUPPLY REVENUE; by James L. Tighe. *Journal,* New England Water Works Association, v. 18, p. 352 (Dec., 1904). (On taxation; refers to the proper valuation of water-works.)

THE NECESSARY ELEMENTS FOR WATER-WORKS VALUATION; by John W. Alvord. *Engineering News,* v. 63, p. 286 (March 10, 1910). (Paper read before the Indiana Sanitary and Water Supply Association; outlines four methods of valuing public utility property which have been upheld by the Courts, and discusses in detail the valuation by reproduction.)

NOTES ON GOING VALUE AND METHODS FOR ITS COMPUTATION; by John W. Alvord. *Proceedings,* American Water Works Association, v. 29, p. 184 (1909). (Contains data on the valuation of water-works properties; ninety-five pages.)

——Abstract. *Engineering-Contracting,* v. 32, p. 95 (Aug. 4, 1909).

THE PRINCIPLES GOVERNING THE VALUATION FOR RATE FIXING PURPOSES of Water Works Under Private Ownership; by Arthur L. Adams. *Journal,* Association of Engineering Societies, v. 36, p. 37 (Feb., 1906). (On the valuation of water-works by the Government; paper read before the Pacific Coast Engineering Congress.)

——*Engineering Record,* v. 52, p. 153 (Aug. 5, 1905).

RATING OF WATER UNDERTAKINGS; by F. J. Bancroft. *Transactions,* British Association of Waterworks Engineers, v. 5, p. 26 (1900). (Discusses the taxation and valuation of English water-works, with reference to the Valuation (Metropolis) Act, 1869.)

——*Journal of Gas Lighting,* v. 76, p. 24 (July 3, 1900).

SOME FUNDAMENTAL CONSIDERATIONS IN THE DETERMINATION OF A REASONable Return for Public Fire Hydrant Service; by Metcalf, Kuichling and Hawley. *Proceedings,* American Water Works Association, v. 31, p. 55 (1911). (Contains data on the valuation of water-works.)

TABLE SHOWING LIFE AND DEPRECIATION OF WATER WORKS PLANTS. *Municipal Journal and Engineer,* v. 24, p. 558 (May 6, 1908).

TOWN WATER-WORKS FROM A FINANCIAL STANDPOINT; by W. L. Hedenberg. *Municipal Engineering,* v. 15, p. 341 (Dec., 1898). (On value of a plant after several years' use; theoretical.)

VALUATION OF WATER WORKS; by Alton D. Adams. *Municipal Journal,* v. 32, p. 942 (June 20, 1913). (Cost of reproduction, value of plant site, distribution system, adequateness of plant, depreciation.)

WATER-WORKS—GENERAL—(*Continued*).

VALUATION OF WATER-WORKS PROPERTIES; by Charles E. Burdick. *Engineering-Contracting*, v. 23, p. 238 (Oct. 23, 1907). (On physical value, depreciation, going value and franchise value; abstract of paper read before Wisconsin League of Municipalities; four pages.)

——*Municipal Journal and Engineer*, v. 23, p. 302 (Sept. 11, 1907).

VALUATION OF WATER-WORKS PROPERTY; by Wynkoop Kiersted. *Transactions*, American Society of Civil Engineers, v. 38, p. 115 (Paper 813. Dec., 1897). (100 pages, including discussion.)

WATER-WORKS APPRAISALS; by Burns and McDonnell. *Engineering Record*, v. 59, p. 616 (May 15, 1909). (Gives brief descriptions of appraisals in nine different cities.)

WATER-WORKS MANAGEMENT AND MAINTENANCE; by Winfred D. Hubbard and Wynkoop Kiersted. John Wiley & Sons, New York, 1907. (Contains a chapter on franchise, water rates, and depreciation.)

WATER-WORKS—SPECIAL CASES.

Antigo, Wis.

***GEORGE W. HILL ET AL. VS. ANTIGO WATER COMPANY**; Submitted June 29, 1908, Decided August 3, 1909. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 3, p. 623. Madison, Wis., 1910. (Gives methods of valuation of water-works plant.)

Appleton, Wis.

***CITY OF APPLETON VS. APPLETON WATER WORKS COMPANY**; Decided May 14, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 215. Madison, Wis., 1911. (Discusses the determination of the present value and reproduction of physical property, value of franchises, etc.)

——The Appleton Water-Works Decision. *Engineering Record*, v. 62, p. 158 (Aug. 6, 1910). (Two pages.)

——Decision of the Railroad Commission of Wisconsin in the Appleton Water-Works Case. *Engineering News*, v. 63, p. 697 (June 16, 1910). (Two pages.)

***IN RE DETERMINING AND FIXING A JUST COMPENSATION TO BE PAID TO** the Appleton Water Works Company by the City of Appleton for the Taking of the Property of Said Company; Submitted Oct. 24, 1910, Decided Dec. 7, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 6, p. 97. Madison, Wis., 1912. (Detailed summary of valuation of physical property of the Appleton Water-Works Co., five-year average, franchise value and going value.)

Ashland Wis.

***CITY OF ASHLAND VS. ASHLAND WATER COMPANY**; Decided Nov. 1, 1909. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 4, p. 273. Madison, Wis., 1910.

Augusta, Me. *See Maine.*

Baltimore, Md.

A RETROSPECT OF AN ARBITRATION ON THE VALUE OF A WATER WORKS; by Albert H. Wehr. *Proceedings*, American Water Works Association, v. 26, p. 361 (1906). (The Mayor and City Council of Baltimore against the Baltimore County Water & Electric Co.; shows method and principle applied to appraisal of the property.)

——Abstract. Arbitration on the Value of a Water-Works. *Municipal Engineering*; v. 31, p. 135 (Aug., 1906).

Beloit, Wis.

***CITY OF BELOIT VS. BELOIT WATER, GAS AND ELECTRIC COMPANY**; Decided July 19, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 201. Madison, Wis., 1912. (Includes appraisal of the physical property of the Water-Works Department.)

——Fixing Normal Operating Costs; by Frank A. Newton. *Engineering Record*, v. 65, p. 258 (March 9, 1912). (Comments on the decision of the Wisconsin Railroad Commission in the case of the City of Beloit *vs.* the Beloit Water, Gas & Electric Co.)

Birmingham, Ala.

THE MERIDIAN CASE; by W. J. Milner. *Proceedings*, American Water Works Association, v. 16, p. 204 (1896). (Concerning the appraisal of the Birmingham, Ala., Water-Works, on their purchase by the City.)

Brunswick, Me. *See Maine.*

WATER-WORKS—SPECIAL CASES—(*Continued*).

California.

*UNIFORM CLASSIFICATION OF ACCOUNTS FOR WATER CORPORATIONS PRESCRIBED by the Railroad Commission of the State of California; Adopted Oct. 23, 1912; Effective Jan. 1, 1913. Sacramento, 1912.

Chicago, Ill.

THIRTY-FOURTH ANNUAL REPORT OF THE DEPARTMENT OF PUBLIC WORKS to the City Council of the City of Chicago for the Year Ending Dec. 31, 1909. pp. 320, 321. Chicago, 1911. (Gives tables of appraisement of water-works equipment and percentage of depreciation.)

Chippewa Falls, Wis.

*T. J. CUNNINGHAM ET AL. VS. CHIPPEWA FALLS WATER WORKS AND LIGHTING Company: In Re Investigation by the Railroad Commission of Wisconsin of Rates Charged by the Chippewa Falls Water Works and Lighting Company; In Re Valuation of the Property of the Chippewa Falls Water Works and Lighting Company; Decided June 14, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 302. Madison, Wis., 1911. (Consideration of factors affecting plant value, going value, working capital and accruing depreciation.)

Contra Costa, Cal.

VALUATION OF THE WORKS OF THE CONTRA COSTA WATER COMPANY. *Engineering Record,* v. 43, p. 516 (June 1, 1901). (Review of a decision in a California Court in which the value of the property of a water-works company is placed at two and one-half times its cost.)

Denver, Colo.

REPORT OF THE BOARD OF APPRAISERS AND ARBITRATORS UPON THE PROPerty of the Denver Union Water Company, March 20, 1909. Denver, 1909. (Report made to the Mayor and City Council, giving estimates on real and personal property; forty-six pages.)

Fond du Lac, Wis.

*IN RE DETERMINING AND FIXING JUST COMPENSATION TO BE PAID TO THE Fond du Lac Water Company by the City of Fond du Lac for the Property of Said Company Actually Used and Useful for the Convenience of the Public; Submitted April 15, 1910, Decided Aug. 19, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 482. Madison, Wis., 1911. (Discusses the original value of plant depreciation, etc.)

——Valuation of the Fond du Lac Water-works. *Engineering Record,* v. 62, p. 275 (Sept. 2, 1910). (Reviews the principles on which the Railroad Commission of Wisconsin based its findings; two pages.)

Freeport, Ill.

A COMPARATIVE STUDY OF THE FOUR PRINCIPAL METHODS OF APPRAISING the Value of Public Utilities with Special Reference to the Valuation of the Freeport (Ill.), Water Works Properties. *Engineering and Contracting,* v. 38, pp. 368, 396 (Oct. 2, 9, 1912). (The original cost method; the commercial or comparative method; the value of a plant as found by the market value of its securities; and the cost of reproducing the property at the present normal prices of material and labor.)

Gloucester, Mass.

THE APPRAISAL OF THE GLOUCESTER WATER-WORKS. *Engineering Record,* v. 40, p. 264 (Aug. 19, 1898). (A review of the principles adopted by the commissioners appointed to hear testimony and appraise the value of a private water company.)

——Appraisal of the Gloucester Water-Works. *Engineering Record,* v. 44, p. 121 (Aug. 10, 1901). (A decision of the Supreme Judicial Court of the Commonwealth of Massachusetts on the valuation of water-works.)

Great Britain.

THE VALUATION OF WATER-WORKS UNDERTAKINGS ON TRANSFER TO MUnicipal Authorities; by E. J. Silcock. *Surveyor and Municipal and County Engineer,* v. 43, p. 920 (June 13, 1913). (Paper read before the Institution of Water Engineers; general principles on valuation of water-works property in Great Britain.)

Indianapolis, Ind.

REPORT OF THE BOARD OF PUBLIC WORKS, INDIANAPOLIS, 1898, p. 13. Indianapolis, 1898. (Contains report upon the physical condition and value of the property of the Indianapolis Water Co.)

WATER-WORKS—SPECIAL CASES—(*Continued*).

Janesville, Wis.

*CITY OF JANESVILLE VS. JANESVILLE WATER COMPANY; Decided August 17, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 628. Madison, Wis., 1912. (Contains summary showing value of each class of property in revised valuation of physical property brought down to Jan. 1, 1910.)

Kansas. See Missouri.

Knoxville, Tenn.

THE KNOXVILLE WATER CASE. *Electric Railway Journal,* v. 33, p. 280 (Feb. 13, 1909). (Decision of U. S. Supreme Court; contains material on valuation and depreciation; one and one-half pages.)

——Editorial. Depreciation of Plant and Fair Rates for Public Service. *Engineering Record,* v. 59, p. 197 (Feb. 20, 1909). (Comments on the decisions of the Supreme Court in the Knoxville water case.)

PROVISION FOR DEPRECIATION BY PUBLIC UTILITY CORPORATIONS. (Letter); by W. H. Winslow. *Engineering News,* v. 61, p. 251 (March 4, 1909). (Comment on decision of the Supreme Court in the Knoxville water case.)

SUPREME COURT DECISION IN THE KNOXVILLE WATER CASE. *Electrical World,* v. 53, p. 386 (Feb. 11, 1909). (Two pages.)

VALUATION OF PUBLIC UTILITY PROPERTIES. *Electrical World,* v. 53, pp. 509, 692, 740, 928, 1036 (Feb. 25, March 18, 25, April 15, 29, 1909). (Discussion between W. H. Winslow and F. E. Haskell in relation to the reasoning of the U. S. Supreme Court as to depreciation, in the Knoxville water case.)

Lake Forest, Ill.

OUTLINE OF A CONTRACT BETWEEN A CITY AND A WATER-WORKS COMpany. *Municipal Engineering,* v. 36, p. 247 (April, 1909). (Ordinance rejected by City Council of Lake Forest, Ill.; included provision for valuation of property according to the British plan.)

London, England.

LONDON WATER-WORKS ACQUIREMENT; by R. Price-Williams. *Engineering,* v. 77, p. 544 (April 15, 1904). (Concerning the report of the Court of Arbitration on the valuation of eight water companies acquired by the Water Board.)

Los Angeles, Cal.

ALLOWANCES FOR DEPRECIATION IN THE PIPE SYSTEM OF THE LOS ANgeles Water-Works. *Engineering News,* v. 41, p. 283 (May 4, 1899). (Gives total valuation of the distribution system and details of depreciation established for different varieties of pipe.)

Madison, Wis.

*CHRISTIAN DICK ET AL. VS. BOARD OF WATER COMMISSIONERS OF THE City of Madison, In Re Valuation of the Plant of the Madison Municipal Water Works; Decided Nov. 11, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 5, p. 731. Madison, Wis., 1911. (A valuation of the Madison water plant undertaken by the Engineers of the Commission.)

——The Method of Making Water Rates Ordered in Wisconsin. *Engineering Record,* v. 63, p. 28 (Jan. 7, 1911). (Valuation of the water-works of Madison, Wis., to assist in determining rates.)

Maine.

A COURT RULING ON THE BASIS FOR APPRAISAL OF A WATER-WORKS PLANT. *Engineering News,* v. 49, p. 75 (Jan. 22, 1903). (Contains instructions to the appraisers appointed to fix the price of water-works property at Waterville, Fairfield, Winslow, Benton, and Mt. Vernon, in Maine, to be taken by the Kennebec Water District.)

MAINE WATER DISTRICTS AND APPRAISERS; by Harvey D. Eaton. *Journal,* New England Water Works Association, v. 19, p. 147 (June, 1905). (Provisions of an act providing for the incorporation of the Kennebec Water District and the appraisal of the water-works of Waterville and Augusta, Me.)

NEW FEATURE IN WATER-WORKS APPRAISALS. (Editorial.) *Engineering News,* v. 54, p. 147 (Aug. 10, 1905). (Considers the relation between the Waterville and Augusta appraisals and the quality of the water supply.)

WATER WORKS VALUATION AND FAIR RATES IN THE LIGHT OF THE MAINE Supreme Court Decisions in the Waterville and Brunswick Cases; by Leonard Metcalf. *Transactions,* American Society of Civil Engineers, v. 64, p. 1 (Paper 1105. Sept., 1909). (111 pages, including discussion.)

——Editorial. Fair Rates for Water Service. *Municipal Engineering,* v. 36, p. 29 (Jan., 1909).

WATER-WORKS—SPECIAL CASES—(*Continued*).

Manitowoc, Wis.

***IN RE DETERMINING AND FIXING A JUST COMPENSATION TO BE PAID TO** the Manitowoc Water Works Company by the City of Manitowoc for the Property of Said Company Actually Used and Useful to the Public; Submitted March 15, 1911, Decided June 27, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 74. Madison, Wis., 1912. (Valuation of physical property and going value of the plant.)

VALUATION OF MANITOWOC WATER-WORKS. *Engineering Record,* v. 64, p. 40 (July 8, 1911). (Valuation made by the Wisconsin Railroad Commission.)

Marinette, Wis.

***CITY OF MARINETTE VS. CITY WATER COMPANY OF MARINETTE;** Decided Dec. 14, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 8, p. 334. Madison, Wis., 1912. (The physical value of the property, the going value, and the apportionment of value between general and fire service.)

Missouri.

WATER-WORKS APPRAISEMENTS IN MISSOURI AND KANSAS; by Clinton S. Burns. *Engineering News,* v. 62, p. 115 (July 29, 1909). (A brief description of nine appraisements made for municipalities; one page.)

Mobile, Ala.

THE MOBILE WATER-WORKS APPRAISAL. (Letter); by J. M. Hazlehurst. *Engineering News,* v. 49, p. 412 (May 7, 1903).

WATER-WORKS APPRAISAL AT MOBILE, ALA. *Engineering News,* v. 49, p. 359 (April 23, 1903). (Report of Committee appointed by the City, in view of the purchase of the water-works by the City.)

——Editorial. *Engineering News,* v. 49, p. 368 (April 23, 1903).

New York, N. Y.

ACQUISITION BY NEW YORK CITY OF THE LARGER OF TWO WATER SYSTEMS of Staten Island; by Louis L. Tribus. *Proceedings,* American Water Works Association, v. 29, p. 557 (1909). (Estimates on the water-works by a commission of engineers giving physical values, going value, etc.)

Oconto, Wis.

***IN RE APPLICATION OF THE OCONTO CITY WATER SUPPLY COMPANY FOR** Valuation of Its Property and Other Relief; Submitted Feb. 10, 1911, Decided Aug. 7, 1911. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 7, p. 497. Madison, Wis., 1912. (Gives tentative and final valuation of the physical property of the plant.)

Omaha, Nebr.

SOME RULES FOR WATER-WORKS APPRAISAL: OPINION OF U. S. SUPREME Court Delivered by Judge Lurton, in City of Omaha *vs.* Omaha Water Company. *Engineering Record,* v. 62, p. 293 (Sept. 10, 1910). (Two and one-half pages.)

Peoria, Ill.

REPORT TO THE MAYOR AND CITY COUNCIL ON WATER RATES FOR THE Plant Belonging to the Peoria Water Works Co., Peoria, Ill.; by Benezette and C. B. Williams. Peoria, 1910. (Treats of controlling principles of public utility values, Court decisions, valuations for sale and rate-making; with a table giving computation of going value of the Peoria Water-Works plant.)

VALUATION OF PHYSICAL PROPERTY OF THE PEORIA WATER CO. WITH A Discussion of Rate-Making and Reasonable Rates. *Engineering and Contracting,* v. 35, p. 192 (Feb. 15, 1911). (Serial includes tables of valuation.)

Quincy, Ill.

THE APPRAISAL OF THE QUINCY WATER-WORKS. *Engineering Record,* v. 50, p. 118 (July 23, 1904). (Report of appraisers appointed by City of Quincy, Ill., in relation to purchase of plant of Quincy Water Co.)

Richmond, Ind.

SOME NOTES AND FIGURES ON THE VALUATION OF THE WATER WORKS Plant at Richmond, Ind.; by Howard A. Dill. *Engineering-Contracting,* v. 33, p. 202 (March 2, 1910). (Gives actual estimates; from a paper before the Indiana Sanitary and Water Supply Association, 1910; one page.)

——Valuation of Water-Works System of Richmond, Ind.; by Howard A. Dill. *Municipal Engineering,* v. 38, p. 410 (June, 1910).

Rockford, Ill.

REPORT ON THE ENLARGEMENT AND EXTENSION OF THE WATER SUPPLY and Distribution System of the City of Rockford, Ill., p. 96; by John W. Alvord, Dabney H. Maury and Daniel W. Mead. Rockford, Ill., 1911. (Contains two and one-half pages on the valuation of the present plant and property.)

WATER-WORKS—SPECIAL CASES—(*Continued*).

San Antonio, Tex.

THE ADJUSTMENT OF WATER RATES. *Engineering Record,* v. 45, p. 127 (Aug. 9, 1902). (Some features of a valuation of the San Antonio, Tex., Water Works Co., made by Chester B. Davis.)

REPORTS ON WATER-WORKS RATES AND VALUATION AT SAN ANTONIO, TEX. *Engineering News,* v. 48, p. 233 (Sept. 25, 1902). (Report made by Chester B. Davis.)

Superior Water, Light & Power Co.

***ESTIMATING THE RATE OF REASONABLE RETURN FOR A PUBLIC UTILITY.** *Engineering and Contracting,* v. 39, p. 482 (April 30, 1913). (Argument of William H. Winslow, Vice-President of the Superior Water, Light & Power Co. before the Wisconsin Railroad Commission, giving analysis of the rate of fair return to be allowed on capital to be invested in public utilities.)

Syracuse, N. Y.

MUNICIPAL ACQUIREMENT OF PRIVATE WATER COMPANY PLANTS, AS Illustrated by the Syracuse, N. Y., Water Company Condemnation by the City; by Stephen E. Babcock. *Proceedings,* American Water Works Association, v. 13, p. 85 (1893). (Discusses actual values assigned in Syracuse Water-Works appraisal.)

——Abstract. Municipal Acquirement of the Plant of the Syracuse Water Co. *Engineering News,* v. 30, p. 207 (Sept. 14, 1893).

Valparaiso, Ind.

THE WATER-WORKS APPRAISAL CASE AT VALPARAISO, IND. (Editorial Note.) *Engineering News,* v. 49, p. 289 (April 2, 1903). (Value decided by the Court; basis of decision.)

Washburn, Wis.

***CITY OF WASHBURN VS. WASHBURN WATER WORKS COMPANY;** Decided Dec. 6, 1910. In Opinions and Decisions of the Railroad Commission of the State of Wisconsin, v. 6, p. 74. Madison, Wis., 1912. (Contains tentative valuation of the physical property of the Company.)

Waterloo, Iowa.

STRANGE CASE OF WATER-WORKS APPRAISAL. (Editorial.) *Engineering Record,* v. 59, p. 502 (April 17, 1909). (Discusses estimates in the valuation of the Waterloo, Iowa, Water-Works made by A. Marston.)

THE VALUATION FOR CITY PURCHASE OF THE PROPERTY OF THE WATERLOO (Ia.) Water-Works Co.; by A. Marston. *Engineering News,* v. 61, p. 424 (April 22, 1909). (General explanation with abstract of explanatory report; one page.)

Waterville, Me. See Maine.

WATER-WORKS—UNVERIFIED REFERENCES.

DEPRECIATION OF WATER WORKS; by John W. Alvord. (Serial.) *Fire and Water Engineering,* v. 34, pp. 104, 144 (Aug. 22, Sept. 12, 1903).

KNOXVILLE WATER CO. VS. CITY OF KNOXVILLE: Report of Special Master; Decision of Judge Clark, U. S. Circuit Court, East. District of Tennessee, Northern Division, Aug. 19, 1904. Jan. 24, 1905.

MEMOIRS OF DECEASED MEMBERS.

BENJAMIN MORGAN HARROD, Past-President, Am. Soc. C. E.*

DIED SEPTEMBER 7TH, 1912.

Benjamin Morgan Harrod was born in New Orleans, La., on February 19th, 1837. His father, Charles Harrod, came to that city in 1809, and made it his home. His mother, Mary Morgan, was a native of Louisiana, and the daughter of Benjamin Morgan who had come from Pennsylvania in the late years of the previous century and engaged in business and planting.

After attending the local schools, he prepared for college at Flushing, N. Y. He afterward entered Harvard University and was graduated in the class of 1856, with a well-formed intention of following a technical or constructive profession.

After a year or more of preparatory study, Mr. Harrod secured employment as Draftsman, in the United States Engineer Office in charge of the construction of forts and light-houses on the Gulf Coast, from the Mississippi River to the Rio Grande River, and subsequently was appointed Assistant Engineer. In 1859 he commenced practice in New Orleans, as an Engineer and Architect.

In April, 1861, he enlisted as a private in the Confederate Army, and was soon commissioned as Lieutenant in an Artillery Regiment, being detailed on engineer duty. He served as Brigade and Division Engineer with the command of Gen. M. L. Smith in the fortification and defense of New Orleans and Vicksburg. When exchanged after the surrender of the latter place, he was commissioned as Captain of Engineer troops in Virginia, was engaged in the defense of Petersburg and Richmond, and followed the fate of that Army to the surrender at Appomattox, in 1865. He then resumed professional practice in New Orleans.

In 1877 Major Harrod was appointed Chief State Engineer of Louisiana, with the late H. B. Richardson and T. S. Hardee, Members, Am. Soc. C. E., as associates, on a board, the principal function of which was the construction of a system of levees to protect the alluvial regions of the State from overflow.

In 1879 he was appointed an Engineer member of the Mississippi River Commission, charged by the United States Government with the survey of the Mississippi River and its tributaries, and the improvement of the main streams from the junction of the Ohio to the head of the Passes, with special reference to confinement and its conservative influence on navigation. The importance of this latter

* Memoir prepared by the following committee: Frank M. Kerr, Sidney F. Lewis, and Arsene Perrilliat, Members, Am. Soc. C. E.

Ben McHarrod

part of the work of the Commission was subsequently recognized by the Government, and the repair and building of levees was made mandatory. It has been one of the most beneficent and remunerative works of the age, affecting the reclamation from overflow of 30 000 sq. miles of territory of unsurpassed fertility, and its development in population, agriculture, commerce, and wealth.

From 1888 to 1892 Major Harrod was City Engineer of New Orleans, and, subsequently, Advisory Engineer for the drainage, sewerage, and water-works systems of that city; and from 1897 to 1902, he had charge of the drainage, both designing and constructing.

In 1903 he was appointed, with Gen. C. W. Raymond and John Bogart, Members, Am. Soc. C. E., as the first United States Delegates to the International Congress of Navigation, held that year at Düsseldorf, Germany, but was prevented by business from attending its sessions.

In 1904 he was appointed a Member of the Panama Canal Commission, and served until 1907, when the type of the canal was determined and the charge of the work was transferred to the present Commission. His subsequent work was as a Consulting Engineer in New Orleans.

He married, in 1865, Miss Harriet Shattuck Uhlhorn, and, in 1883, Miss Eugenia Uhlhorn, both of New Orleans, the latter surviving him.

The degree of LL.D. was conferred on him by Tulane University, in 1906, as a

"Graduate of Harvard University fifty years ago; President of the American Society of Civil Engineers; expert specialist and virile all-round man; friend of Tulane University and of all movements to better this City. In deepening a river and now in cutting an isthmus, his work has ever been to bring men closer in commerce, in friendship, and in mutual helpfulness."

Major Harrod was also a Member of the Association of Harvard Engineers, and its Vice-President in 1909, as well as a Member of the Louisiana Engineering Society.

He was a man of sterling qualities and strong individuality, but at all times generous and kind, and the friend of the worthy young aspirant for advancement in the Profession. Recognized, as he was, as a man of letters, of science, and of pronounced artistic taste, and as a high-toned, patriotic, public-spirited, loyal citizen, his loss will long be felt by the many who in life had the privilege of knowing him and in consequence admiring him.

Major Harrod was elected a Member of the American Society of Civil Engineers on April 4th, 1877. He served as Director from 1892 to 1894, as Vice-President in 1895-96, and as President in 1897.

GEORGE EDWARD GRAY, Hon. M. Am. Soc. C. E.*

DIED JANUARY 1ST, 1913.

George Edward Gray, the son of Joel and Betsey R. Gray, was born at Verona, N. Y., on September 12th, 1818.

After receiving a public school education, he studied civil engineering under Pelatiah Rawson, a pioneer in the Profession in this country.

In August, 1839, Mr. Gray was appointed Resident Engineer on the Black River Canal, in New York State, and remained in that position until 1842. He then served as Assistant Engineer, under the late Allan Campbell, M. Am. Soc. C. E., Chief Engineer, and the late A. W. W. Evans, M. Am. Soc. C. E., Resident Engineer, on the New York and Harlem Railroad until 1847, when he returned to the Black River Canal, serving as Assistant Engineer and, later, as Resident Engineer, on that and the Erie Canal.

In the Spring of 1852, he was appointed Chief Engineer of the Utica and Schenectady and the Mohawk Valley Railroads, and, in 1853, when these and other roads were consolidated to form the New York Central Railroad, he was made Chief Engineer of the latter, remaining in that position until May, 1865. It was during his incumbency as Chief Engineer, and under his immediate direction, that the first wrought-iron truss bridges in the United States were built on the line of this railroad. He also acted as Chief Engineer on the construction of the Hudson River Bridge, at Albany, N. Y., from 1860 to May, 1865.

In 1865 Mr. Gray was made Consulting Engineer of the Central Pacific Railroad which, with the Union Pacific Railroad, completed the first transcontinental railroad in the United States. Mr. Gray continued in this position until 1871, when he was employed as Chief Engineer by the Southern Pacific Railroad, and in this capacity had charge of the construction of many miles of that road and of the various other railroads comprising the Southern Pacific System.

In 1885, he retired from railroad work and engaged in consulting practice at his home in Elmwood Park, Berkeley, Cal., until his death on January 1st, 1913.

Mr. Gray was held in affection and respect by all who knew him and his work. He was a member of many technical and scientific associations, including the Institution of Civil Engineers of Great Britain, the California Academy of Sciences, the San Francisco Association of the American Society of Civil Engineers, etc., and served as a Trustee of the Leland Stanford, Jr., University for many years.

Mr. Gray was elected a Member of the American Society of Civil Engineers on July 2d, 1873, and an Honorary Member on June 5th, 1894.

* Memoir prepared by the Secretary from material on file at the Society House.

PETER SUTHER ARCHIBALD, M. Am. Soc. C. E.*

DIED MARCH 16TH, 1913.

Peter Suther Archibald, the son of William and Elizabeth (Blair) Archibald, was born at Truro, Nova Scotia, on March 21st, 1848, and received his education in the Truro Model and Normal Schools.

Mr. Archibald began his professional career as a Rodman on the preliminary surveys for the Intercolonial Railway in September, 1867, under *Sir* Sandford Fleming, M. Am. Soc. C. E., and, during the construction of the road and its subsequent operation, he rose, by sheer ability and application, from one position to another, until, in 1879, he became Chief Engineer of the railroad which, as a boy, he had helped to construct. This position he held until his resignation, in 1898, to take up private practice as a Consulting Engineer.

Mr. Archibald was Consulting Engineer of the St. John Cantilever Bridge and Railway Extension Company, by special permission of the Minister of Railways. In 1894 he was appointed a member of the Royal Commission to inquire into freight rates in Manitoba and the Northwest Territories.

In 1902 he was made Chairman of an Arbitration Tribunal between Contractor Reid and the Newfoundland Government, which involved a $2 000 000 claim; and, later, he was Chairman of the Arbitration Board to settle the question of the Newfoundland telegraph lines. He was also employed as an expert in disputed matters between the Clergue Syndicate and its contractors on railway construction.

In 1905 Mr. Archibald was Contractor's Engineer for Mr. A. E. Trites, on the construction of the Northern Maine Seaport Railroad, now a part of the Bangor and Aroostook Railroad. In 1907 he was appointed a member of the Conciliation Board *re* the Cumberland Coal and Railway Company, under the Industrial Disputes Act, and in 1908 he was made a Commissioner by the New Brunswick Government to manage the New Brunswick Central Railway.

Mr. Archibald's capabilities as a Consulting Engineer were so widely recognized that, after taking up private practice, his services and advice were frequently sought in cases of special importance, as well as in many smaller projects for railways, bridges, and water supplies.

Mr. Archibald was a wide reader; a man of calm, even temperament, a true friend, and a gentleman who possessed the confidence and respect of those who had the pleasure of his acquaintance. By his

*Memoir prepared by William B. MacKenzie, Esq., Right-of-Way and Lease Agent, Intercolonial Railway of Canada, Moncton, N. B., Canada.

ever-ready desire to assist those in need, he secured not only the respect of his professional brethren, but the admiration and loyalty of his subordinates.

In April, 1874, Mr. Archibald was married to Miss Clara G. Lindsay, the daughter of Mr. F. S. Lindsay, of Rockland, Me.

He was a Member and Councillor of the Canadian Society of Civil Engineers, and a Lieutenant in the 73d Volunteer Battalion.

Mr. Archibald was elected a Member of the American Society of Civil Engineers on January 7th, 1885.

THOMAS CHALKLEY JAMES BAILY, JR., M. Am. Soc. C. E.*

DIED DECEMBER 6TH, 1912.

Thomas Chalkley James Baily, Jr., was born in Boston, Mass., on December 20th, 1867. His father, Major Thomas Chalkley James Baily, U. S. A., served throughout the Civil War, being commissioned First Lieutenant on May 14th, 1861, and retiring with the rank of Major, on August 18th, 1865. His mother, Carrie E. Todd, a native of Augusta, Me., was of Puritan ancestry, and descended from the earliest New England settlers.

After attending the local schools, Mr. Baily prepared for college at the Newark School, Newark, Del. He afterward entered Lehigh University, and was graduated in the Class of 1890, with high honors, and with the degree of Civil Engineer.

For two years after his graduation, he was a member of the Engineer Corps of the Erie Railroad, stationed at New Castle, Pa., resigning to accept a position on the Mississippi River improvement, where he was engaged in examinations and surveys of the mouth of the Mississippi. In 1893, he entered the Engineer Department of the District of Columbia, first as Draftsman and then as Assistant Engineer in the Highway Division. In 1908, he was appointed Principal Assistant Engineer in the Sewer Department of the District of Columbia; two years later he was made Assistant Engineer in charge of Street Extensions; and, in 1910, he was appointed Engineer of Bridges for the District of Columbia.

Mr. Baily was in charge of the construction of the Anacostia River Bridge, and, as Engineer of Bridges, he designed and built a number of important structures, among them the Cedar Street Subway at Takoma Park, D. C., and the Belmont Street Improvement. At the time of his death, he had just completed the plans and specifications for the Q Street Viaduct, crossing Rock Creek Valley at Washington.

He assisted in the design and construction of the Edmonton Avenue Bridge at Baltimore, Md., and collaborated with W. J. Douglas, M. Am. Soc. C. E., in writing the chapters on Masonry and Timber Structures for the "American Civil Engineers' Pocket Book," edited by Mansfield Merriman, M. Am. Soc. C. E.

At the time of his death, Mr. Baily was making investigations of the Harbor of Prógresso, Yucatan, for Barclay Parsons and Klapp, Consulting Engineers. He died suddenly in the City of Mexico, on December 6th, 1912.

* Memoir prepared by Asa E. Phillips, M. Am. Soc. C. E.

In 1894, he was married to Miss Mary Rodgers, of Orange, N. J., who, with one daughter and one son, survives him.

Mr. Baily was a thorough and painstaking engineer, and an exceptionally devoted student of his Profession. He had a kind and genial nature, and was esteemed and admired by all who knew him. He was a devoted father, a faithful friend, and a good citizen.

Mr. Baily was elected a Member of the American Society of Civil Engineers on October 4th, 1905.

ANDREW BELL, M. Am. Soc. C. E.*

DIED OCTOBER 22D, 1912.

Andrew Bell was born at Toronto, Ont., Canada, on December 21st, 1835. His father, the late Rev. Andrew Bell, was the eldest son of the Rev. William Bell, the pioneer clergyman of Perth, Ont., and the surrounding country. His mother was the eldest daughter of the late Col. E. W. Thomson, of Toronto, a cousin of Baron Sydenham, who was Governor of Upper Canada at the time of consolidation of the provinces.

Mr. Bell was educated at Queen's College, from which he was graduated in 1853 with the degree of B. A. Immediately after his graduation he was articled to Mr. William Syles as a student of Civil Engineering.

In February, 1854, he was appointed Rodman on the survey of the Brockville and Ottawa Railway (now a part of the Canadian Pacific Railway) from Smith's Falls to Carleton Place, and to Perth. This work was discontinued in 1855, and Mr. Bell was engaged in making surveys, under K. Tully, Esq., for the proposed canal from Toronto to Georgian Bay.

In July, 1856, he was appointed Resident Engineer on the Brockville and Ottawa Railway, which position he retained until 1859, having been engaged in the following work: In 1856 he surveyed and located the line from Brockville to Smith's Falls; in October of the same year, he was sent to Almonte, Ont., and placed in charge of the construction, under the late Alphonse Brooks, from Carleton Place to Pakenham; and from 1857 to 1859, he was in charge of the surveys from Sand Point to Pembroke and of the completion of the construction between Carleton Place and Almonte.

Between 1864 and 1869, Mr. Bell had charge of the construction of the large woolen mills at Almonte and Cornwall, Ont., being engaged, at the same time, in general private practice. In 1869 he made a hydraulic survey between Lake Superior and Lake Winnipeg for the Dominion Government.

In June, 1870, he was appointed Resident Engineer on the enlargement of the Grenville Canal, remaining in this position until June, 1872, when he became Resident Engineer on the construction of the Carillon Canal Dam and Slide. After the completion of this work, in 1886, Mr. Bell removed to Almonte, Ont., where, until his death, he was engaged in private practice as a Civil Engineer, Land Surveyor, and Architect. While engaged on a survey of the country east of

* Memoir prepared by the Secretary from information on file at the Society House.

Cochrane, on the Grand Trunk Pacific Railway, he was taken ill and was forced to return to his home in Almonte, where he died on October 22d, 1912.

In 1858, Mr. Bell was married to Miss Marion Rosamond, eldest daughter of the late James Rosamond, of Almonte, who, with five children survives him.

Mr. Bell was elected a Member of the American Society of Civil Engineers on September 5th, 1883. He was also a Member of the Canadian Society of Civil Engineers, and many other technical associations.

DANIEL SEYMOUR BRINSMADE, M. Am. Soc. C. E.*

DIED SEPTEMBER 7TH, 1912.

Daniel Seymour Brinsmade was born at Trumbull, Conn., on February 17th, 1845. He was the youngest son of Daniel Stiles Brinsmade and Catherine Mallette Brinsmade, his family having resided continually for more than 250 years within the limits of the original town of Stratford, Conn.

Mr. Brinsmade received his early education in the public schools of Trumbull and at the Gunnery at Washington, Conn. He then entered the Sheffield Scientific School of Yale University, from which he was graduated in 1870.

Immediately after his graduation, Mr. Brinsmade was appointed Assistant Engineer on the construction of the dam across the Housatonic River at Huntington, Conn., for the Ousatonic Water Company. In 1871, he was made Chief Engineer of the project, and afterward served successively as Secretary and Treasurer. In 1900, he was made President of the Company, which position he held until his death, at Shelton, Conn., after an illness of nearly two years. In 1891, the old dam was destroyed, and Mr. Brinsmade had to rebuild the structures and add many new features in order to meet the conditions which obtained at that time.

He made hydraulic engineering his life study, and, being an authority on the subject, was continually engaged in the construction of water-supply and water-power projects. He was widely known as a Consulting Engineer throughout New England and New York State, and served as a member of a commission appointed by the State of Maine to consider the question of water-power development in that State.

Mr. Brinsmade was a genial, cultured gentleman, whose advice was eagerly sought in business as well as in private life, and whose salient characteristics were honesty and truth. He took great pride in the betterment of his home town, Shelton, Conn., and was closely connected with all phases of activity within the community. He served the Borough in nearly every elective position in its gift. He was also elected to the General Assembly as Representative and State Senator, and, although a staunch Republican, he did his duty in these positions regardless of party.

Mr. Brinsmade was a Director of the Shelton Water Company, a Trustee of the Griffen Hospital, President of the Shelton School Board for nearly twenty-eight years, President of the Plumb Memorial Library Directorate, and a Trustee of the Riverside Cemetery Association, all

* Memoir prepared by the Secretary from information on file at the Society House.

of which institutions he was largely interested in establishing and maintaining. He was also Vice-President of the Home Trust Company, of Derby, Conn., and a Director of the Birmingham National Bank. He was a Member of the Congregational Church, of Shelton, and also prominent as a Mason and an Odd Fellow.

Mr. Brinsmade was elected a Member of the American Society of Civil Engineers on February 1st, 1888. He was also a Charter Member of the Connecticut Society of Civil Engineers and served as its President in 1907.

ALFRED ELLSWORTH CARTER, M. Am. Soc. C. E.*

DIED JUNE 11TH, 1912.

Alfred Ellsworth Carter was born at Blair, Nebr., on April 19th, 1867, of American parents, his ancestry dating back through several generations of pioneer stock. His early education was obtained at the public schools of his home town, and, later, at the University of Nebraska, from which he received the degree of Bachelor of Science, in 1900. In 1902 he entered Columbia University, New York City, and was graduated in 1904 with the degree of Civil Engineer.

Mr. Carter's early experience was gained while earning his way through college. In the the early Nineties he was in the employ of the Chicago and Northwestern Railway Company, in Fremont, Elkhorn and Missouri Valley Railroad activities, holding successively the positions of Chainman, Rodman, and Transitman, on miscellaneous surveys in Nebraska and the Black Hills of South Dakota. From August, 1897, to March, 1899, he was Assistant Engineer on the construction of a hydro-electric power dam at Divide, Mont., an impounding reservoir dam adjacent to Butte, Mont., and one of the first wood stave and riveted steel pipe lines for the Montana Power Company.

Following the Spanish-American War, from October, 1900, to October, 1902, Mr. Carter was Assistant Engineer in charge of detailed designing of sewers and pumping stations for two sections of the marginal sewer system of Havana, Cuba, being associated with Samuel M. Gray, M. Am. Soc. C. E., Consulting Engineer, the work being done by the Department of Sewers, under Military Government, William M. Black, M. Am. Soc. C. E., Colonel, Corps of Engineers, U. S. A., being in general charge at Havana.

From January, 1905, to the time of his death, Mr. Carter was in the employ of the Rapid Transit Subway Construction Company, Contractors, of New York City, as Assistant Engineer, until 1908, in charge of tunnel alignment, check surveys, track-laying, and driving reinforced concrete piles, on the construction of the East River Tunnel of the Rapid Transit Railroad; then Resident Engineer in charge of construction of the Bowling Green Shuttle Station, and the Subway station extensions at the Fulton Street, Wall Street, Bowling Green, Borough Hall, and Atlantic Avenue Stations of the Interborough Rapid Transit Company. He was also engaged in reporting on extra claims of the Sub-contractor on the East River Tunnel.

At the time Mr. Carter became engaged on the work of the East River Tunnel, the Brooklyn tubes were just entering the river sec-

* Memoir prepared by George H. Pegram, M. Am. Soc. C. E.

tion and the Manhattan tubes had not emerged from the rock. He was employed continuously on this work until its completion in January, 1908. It has been described as one of the most difficult pieces of engineering work ever accomplished. Mr. Carter's position as Assistant Engineer imposed great responsibilities on him. He was in charge of the delicate operations of sinking piles through the bottom of the tubes to rock. The character of the work and the financial failure of the Sub-contractor, during the construction of the tunnel, made the accounting unusually complicated. The patience and fidelity with which Mr. Carter worked in checking the claims of the Sub-contractor and the skill and judgment evinced in his reports are remarkable. It was a work of great labor and uncongenial to an Engineer, but his familiarity with the construction forced it on him. His engineering work had been above criticism, but this work was almost above praise.

Subsequently, Mr. Carter was put in charge, as Resident Engineer, of the construction of Bowling Green Shuttle Station and the lengthening of the Subway stations from Fulton Street, Manhattan, to Atlantic Avenue, Brooklyn. Like his East River Tunnel experience, this work was of the most difficult character. The continuous operation of trains, the congested street traffic, the numerous sub-surface structures which interfered with the work, such as sewers, water pipes, electric subways, and the foundations of buildings, made it always a delicate task.

The work of extending the Borough Hall Station in Brooklyn, for which Mr. Carter designed the shoring and directed the work for the Construction Company, was especially difficult. This station was built of reinforced concrete which was exceptionally difficult to remove. At this point there are three tracks in the Subway, with cross-overs, and on the surface of the street there is a junction of two tracks in Court Street and two tracks in Fulton Street. Both side-walls of the Subway, for a length of 135 ft., were entirely removed, and its roof with 7 ft. of cover, together with street structures, etc., was supported on timber. Three columns of the Elevated Railroad in Brooklyn were temporarily supported over the work and the foundations renewed; the cast-iron pipes and the gate-chambers of the high-pressure water mains were supported and reconstructed at an especial hazard to the work. In addition, the portico of the County Court House, weighing more than 1 000 tons, a structure with four large granite columns, thus having little transverse stiffness, was temporarily supported, and the foundations were carried 12 ft. deeper by masonry underpinning. This was done without the slightest show of crack or any measurable settlement of the portico. All this work was in sand and Mr. Carter was continually obliged to render it safe against the breakage of water pipes or the unusual flood of storm-water.

This work was about completed at the time of his death, which occurred suddenly at his home in New York City on June 11th, 1912, from cerebral hemorrhage.

Mr. Carter was a man of sterling integrity, with the ability for doing hard work well, and accepting and fulfilling growing responsibilities with quietness and efficiency; the consideration he gave to all matters, large or small, entrusted to his care, had won for him the respect of his associates and those who worked under his direction.

In 1904 Mr. Carter was married to Miss Ida C. Messer, of Cleveland, Ohio, who survives him. She is a lady of unusual educational attainments and was able to assist him in his professional work.

He was a Member of Columbia Chapter (Kappa) of the Society of the Sigma Xi, and a Member of Washington Lodge No. 21, F. & A. M., of Blair, Nebr.

Mr. Carter was elected an Associate Member of the American Society of Civil Engineers, on June 4th, 1902, and a Member on April 4th, 1911.

PHILIP HENRY COOMBS, M. Am. Soc. C. E.*

Died March 6th, 1913.

Philip Henry Coombs, the son of P. H. and Sarah Forbes (Woodhull) Coombs, was born at Bangor, Me., on December 24th, 1856. He was educated in the public schools of that place, and in 1875 entered the office of his uncle, the late Frederick H. Coombs, who was then City Engineer of Bangor, as his Principal Assistant, remaining in this position until 1883.

In November, 1883, Mr. Coombs was elected City Engineer and Superintendent of Sewers of Bangor, which office he held continuously until March, 1893. He occupied the same office in 1894, 1899, and from 1901 to 1905, inclusive. He also served as City Engineer for Brewer, Me. During his incumbency as City Engineer, Mr. Coombs had charge of the construction of many miles of sewers, and thereby became an authority on the plan of the City of Bangor, its streets, and sewerage system, a knowledge which proved of great value during his long period of service as a public official.

As City Engineer, he also made the plans, specifications, and contracts, and had direct charge of the construction of the masonry pier and abutments for Kenduskeag Bridge in 1884 and 1889, and of the masonry pier for the Franklin Street Bridge, in 1885, this latter work having been done jointly by the United States Government and the City of Bangor. He also supervised the erection of the new steel bridge between Bangor and Brewer; the reconstruction and extension of the Bangor Water-Works; and the erection of the Eastern Maine Insane Hospital, as well as the construction of roads and the design of the landscape work.

In addition to the work done for the City of Bangor, Mr. Coombs made surveys and plans for water-works and sewerage systems in many other towns in Maine, including Dexter, Dover, Foxcroft, Newport, and Brewer.

In 1887, he formed a partnership with Thomas W. Baldwin, M. Am. Soc. C. E., under the firm name of Baldwin and Coombs. The firm was engaged in general engineering practice until 1892, when Mr. Baldwin removed from Bangor, and the business was taken over by Mr. Coombs.

Mr. Coombs was widely known as a Consulting Engineer, and had charge of the construction of many large industrial plants, including the pulp mills of the Penobscot Chemical Fibre Company, at Great Works, Me., in 1881-83; the paper and pulp mills of the Eastern Manufacturing Company, at Brewer, in 1889; the Orono Pulp and

* Memoir prepared by the Secretary from material on file at the Society House.

Paper Company, at Basin Mills, Orono, Me.; and the Webster Paper Company, at Orono, Me., in 1890.

In 1888, he had charge of laying out the Bangor Street Railway, the first electric road built in Maine, and one of the first to be operated successfully in the United States. He was also engaged as Engineer on the survey and construction of railroads of several companies centering in Bangor, from 1888 until his death.

In 1901, Mr. Coombs was one of two Commissioners, authorized by the Maine Legislature and appointed by the Penobscot Log Driving Company, on a survey of the West Branch water-shed of the Penobscot River to investigate and determine the present storage, and need of increased storage for log-driving, manufacturing, and other purposes. He remained in the service of the Penobscot Log Driving Company until his death, being engaged chiefly on hydraulic work.

Mr. Coombs was also employed at various times as Engineer in charge of the construction of Chapin Park, Broadway Park, and Summit Park, for the City of Bangor, and for a number of years he was also, as Engineer, in charge of the designs, plans, specifications, etc., of fishways for the Maine Fisheries Commission.

In January, 1883, Mr. Coombs was married to Miss Millie M. Field, of Bangor, Me., who, with two daughters, survives him.

On July 3d, 1911, he was suddenly stricken with what at first was thought to be sunstroke, but which later proved to be paralysis. He had partly recovered his health, and was able to attend to business, but on March 2d, 1913, he had an attack of acute indigestion, from which he died.

Mr. Coombs was a man of pleasant, genial disposition and had a wide circle of friends. He was especially prominent in Masonic affairs, and was also a member of the Order of Odd Fellows.

Mr. Coombs was elected a Member of the American Society of Civil Engineers on March 7th, 1906.

SAMUEL LISPENARD COOPER, M. Am. Soc. C. E.*

DIED MAY 8TH, 1913.

Samuel Lispenard Cooper was born in the City of New York on January 16th, 1858. His father, Samuel Hawes Cooper, was a nephew of Peter Cooper; his mother, Helen Sophia Lispenard was of French Huguenot descent. Mr. Cooper received his early education in the public schools of New York City and then studied at the College of the City of New York and at New York University. He was graduated from the latter institution in 1878 with the degree of Civil Engineer.

For the first five years of his professional career, Mr. Cooper was engaged on the surveys and preliminary work for the New Croton Aqueduct and Reservoirs, and was then placed in charge of the maintenance of the Old Croton Aqueduct, as Resident Engineer. In 1889 he entered the service of the Finance Department of the City of New York, as Supervising Engineer, and continued in this position for the next four years, gaining valuable experience in many directions. When the Department of Public Works of the City of Yonkers was organized, in 1893, Mr. Cooper was placed at its head as Commissioner, and continued in this position for 14 years until Yonkers became a second-class city, in 1908, when, under the reorganization that took place, he was appointed City Engineer, a position he filled until his death.

During 20 years of uninterrupted service with the City of Yonkers, first as Commissioner, and then as City Engineer, Mr. Cooper was responsible for the entire street system of the City. The City Pier and Recreation Pavilion were constructed according to his plans, and the City Hall Building was erected under his supervision. He also had much to do with making plans for the parks, and acted as Consulting Engineer to the Bronx Valley Sewer Commission and in connection with other enterprises.

In 1885, Mr. Cooper was married to Elizabeth Underhill, daughter of the Rev. George H. Goodsell, D.D., and niece of Bishop Daniel A. Goodsell, of the Methodist Episcopal Church, who with three children, Samuel G. Cooper, of Portland, Ore., Mrs. Raymond C. Rose, and Mary L. Cooper, survive him.

Mr. Cooper was an engineer of high ability, a man of sterling character and of indefatigable industry. Performing his duties with absolute honesty, he could not avoid making enemies, but he won the esteem of all good citizens.

During the last two years of his life Mr. Cooper suffered from an incurable disease. He bore this misfortune with great fortitude, and attended faithfully to his official duties to the last, even when scarcely able to leave his home.

* Memoir prepared by Edward Wegmann, M. Am. Soc. C. E.

On May 8th, 1913, he was relieved from his suffering. His funeral was attended by the Mayor, Aldermen, and other City officials, past and present, of Yonkers, and by a large number of citizens who showed by their presence the high esteem in which they held their former City Engineer.

Mr. Cooper was a member of the New York University Alumni Association, Delta Upsilon Fraternity, a Director of the Hollywood Inn, a member of the Nepperhan Lodge of Free Masons, Terrace City Chapter of Royal Arch Masons, of the Elks, of the City Club, and of the Central Methodist Episcopal Church of Yonkers.

Mr. Cooper was elected a Member of the American Society of Civil Engineers on February 6th, 1889.

GEORGE BLINN FRANCIS, M. Am. Soc. C. E.*

DIED JUNE 9TH, 1913.

George Blinn Francis, the son of Blinn and Lucy (Hart) Francis, was born at West Hartford, Conn., on January 31st, 1857. He received his early education in the public schools of his native town and at the Hartford High School, from which he was graduated at the age of 17.

On April 6th, 1874, he entered the employ of the Providence, R. I., Water-Works, as a student, and served for a period of three years in the Engineering Corps, during the construction of the Pettaconset Pumping Station. In March, 1877, he was transferred to the City Engineer's Department, which had just been consolidated with the Water-Works, where he served for the next 4½ years as Assistant Engineer, being assigned to the corps in charge of the "Brook Street District" improvements, and, on the completion of that work, to the Water Department.

In 1881 he joined the Engineering Department of the New York, West Shore and Buffalo Railroad and was assigned to the Ulster Division, in charge of a Hydrographic Corps between Fort Montgomery—at the southern entrance to the Highlands—and Poughkeepsie; later, he was made Assistant Engineer in charge of 4 miles of construction on this Division.

Early in 1883 Mr. Francis went to the Pacific Coast with a party selected to make surveys for a relocation of the Oregon Railway and Navigation Company's line between Portland and The Dalles, through the Cascade Mountains. In the spring of that year he was transferred to Portland as Assistant Engineer in charge of the construction of a large freight terminal, for the Northern Pacific Terminal Company, on the Willamette River, opposite Portland, including the construction of the car-ferry inclines, in connection with the transfer of passengers, for cars across the river, which were to be built in thirty days in order to be in readiness for the transfer of the famous "Gold Spike" party over the Northern Pacific Railway, in September, 1883.

On the completion of this work he returned East and for the next two years was employed on general railroad work—design, construction, and maintenance—including the preparation of plans for harbor works under the direction of the late James B. Eads, M. Am. Soc. C. E., in connection with his scheme for a ship railroad at Tehuantepec, and as Assistant Engineer, on the Hudson River Division (Weehawken to Albany), of the New York, West Shore and Buffalo Railroad, on maintenance of way.

* Memoir prepared by John F. Wallace, Past-President, Am. Soc. C. E., and John W. Ellis, M. Am. Soc. C. E.

In the fall of 1884, Mr. Francis resigned the latter position to accept that of Engineer of the Portland Construction Company, and as such had charge of the design of warehouses, docks, floating pile-driver, coal-pockets, bridges, floating dry-dock, etc. Subsequently, he became Resident Engineer on the construction of the Yakima Division of the Northern Pacific Railroad, returning East on the completion of this work. For the next three years he served in various positions on general railroad work, both design and construction, as follows: In the Engineer Corps on the Blue Mountain and Kittatinny Mountain Tunnels, on the South Pennsylvania Railroad, at Roxbury, Pa.; with the New Jersey Junction Railway, on field location from Weehawken to Bayonne, afterward having charge of the location and construction through Jersey City; and as Division Engineer on general railroad work on the Western Division of the New York Central and Hudson River Railroad, with headquarters at Rochester, N. Y.

On May 1st, 1887, he returned to Providence, R. I., as Principal Assistant Engineer of the New York, Providence and Boston Railroad, remaining in that position until March, 1892, when he was appointed Resident Engineer of the New York, Providence and Boston and Old Colony Railroad Terminal Company (subsequently controlled through lease by the New York, New Haven and Hartford Railroad Company), designing and constructing the Providence Terminal.

On July 1st, 1896, he became Resident and Acting Chief Engineer of the Boston Terminal Company, designing and constructing the South Terminal Station, in Boston, one of the largest passenger terminals in the world. With such care, foresight, and skill were the details of this intricate problem worked out that no important change has ever been made in the layout. The study and experience derived from the successful solution of these two important engineering works led Mr. Francis to specialize in this branch of the Profession, in which field he was recognized subsequently as a leading authority.

In the spring of 1899 he designed and supervised the construction of the foundations of the Kingsbridge Power Station of the Third Avenue Railroad, in New York City.

On the completion of the Boston Terminal, he became Chief Engineer of the Providence Street Railway System, and from February, 1900, to June, 1902, served during a very active period of construction and reconstruction of the Company's urban and interurban lines, building a new generating station, repair shops, car barns, etc.

Mr. Francis resigned to accept a position as head of the Civil Engineering Department of Westinghouse, Church, Kerr and Company, which Company had been retained as Engineers in connection with the new Pennsylvania Terminal in Manhattan, and, during the next eight years, the important civil engineering features of this work were under his supervision.

Some other terminal problems which received his attention are: The proposed Union Station at Toronto, Ontario; the proposed passenger and freight terminal, at Chicago, for the Chicago and Western Indiana Railroad; the Winnipeg Terminal, Grand Trunk Pacific Railway; Vancouver, B. C., Terminal, Canadian Pacific Railway; the proposed freight terminal, Canadian Pacific Railway, at Toronto; the proposed passenger terminal for the City of Buffalo; the rearrangement of the track layout, St. Louis Terminal; and many smaller terminals, both for passengers and freight, in the United States and Canada. As a member of the Railroad Committee of the Merchants Association of New York City, he rendered valuable assistance in the preparation of its report on "The West Side Improvements," relating to railroad and terminal facilities. He thrice visited Europe for observation and study of the large terminals in England and on the Continent.

Some of the many engineering problems which engaged his attention during his connection with Westinghouse, Church, Kerr and Company, were as follows:

Supervision of the construction of two high-speed interurban electric roads, one in the Lackawanna Valley, between Scranton and Wilkes-Barre, Pa., and the other in the Ohio Valley between Beaver, Pa., and Steubenville, Ohio; general supervision of the reconnaissance, location, and plans for a proposed high-speed electric road between Paterson, N. J., and Times Square, New York City; a proposed electric road between Rochester and Syracuse, N. Y., and a proposed line between Baltimore, Frederick, and Hagerstown, Md.; Consulting Engineer on hydro-electric development for the Atlanta Water and Electric Power Company; Consulting Engineer, Shut-off Dam, Charles River Basin; Consulting Engineer, deep foundations, Providence Journal Building; Consulting Engineer, East Side Easy Grade Street, Providence, R. I.; Associate Consulting Engineer with Gustav Lindenthal, M. Am. Soc. C. E., on reconstruction of Locust Point Pier for the Baltimore and Ohio Railroad, at Baltimore, Md.; Consulting Engineer, heavy reinforced concrete bridge, designed for Bush and Gunpowder River Bridges, Pennsylvania Railroad; and investigations and reports on various water-power and irrigation projects in the South and West.

Mr. Francis was a member of the Institution of Civil Engineers of Great Britain; the Boston Society of Civil Engineers; the Canadian Society of Civil Engineers; the New York Railroad Club; and the Boston Engineers' Club. He was very active in the affairs of the Boston Society of Civil Engineers, having been the first non-resident member to be honored with the Presidency, which office he filled from March 17th, 1910, to March 16th, 1911.

He was Past-Master of What Cheer Lodge No. 21; Member of the Providence Royal Arch Chapter No. 1; Providence Council No. 1, R. and S. M.; Calvary Commandery No. 13, K. T., and of Rhode Island Consistory, A. A. S. Rite, N. M. J.

He was a frequent contributor to the technical press and to the publications of various engineering societies. In 1906, with W. F. Dennis, M. Am. Soc. C. E., he was awarded the Thomas Fitch Rowland Prize by the Society, for the paper entitled "The Scranton Tunnel of the Lackawanna and Wyoming Valley Railroad."* Other papers contributed to the Society by Mr. Francis are: "The South Terminal Station, Boston, Mass.," "Electric Railways in the Ohio Valley between Steubenville, Ohio, and Vanport, Pennsylvania," and "The New York Tunnel Extension of the Pennsylvania Railroad. Certain Engineering Structures of the New York Terminal Area."†

He was awarded a gold medal at the Paris Universal Exposition in 1900, in connection with the exhibit of the Boston Terminal Company; and in 1906, he received the Honorary Degree of A. M. from Brown University.

Mr. Francis was a man of keen perception, retentive memory, and rare judgment. He was quick to grasp the essentials of a problem, and brought to its solution a trained mind and a wide and varied experience. His genial disposition, democratic spirit, and kindly courtesy won for him the unfailing loyalty, unfeigned affection, and unwavering devotion of all who ever worked for or with him. His death is keenly felt by a host of friends and business acquaintances, and is a distinct loss to the Profession which he loved, in which his life found its expression, and for which he zealously labored.

In April, 1882, Mr. Francis was married to Florence Louise Greene, of Providence, R. I., who, with one son, George B. Francis, Jr., survives him.

Mr. Francis was elected a Junior of the American Society of Civil Engineers on September 5th, 1883, and a Member on November 7th, 1888. He had also served for two years as a Member of the Nominating Committee.

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* *Transactions*, Am. Soc. C. E., Vol. LVI, p. 219.

† Joint author with Joseph H. O'Brien, M. Am. Soc. C. E.

CHARLES LEWIS HARRISON, M. Am. Soc. C. E.*

DIED SEPTEMBER 14TH, 1912.

Charles Lewis Harrison was born on his father's farm at Auxvasse, in Callaway County, Missouri, on March 5th, 1857, and was one of a family of twelve, having seven sisters and four brothers. He was descended from the families of Harrison and Crockett, of Virginia, his grandfather, Major John Harrison, emigrating to Missouri in 1817, and being one of the earliest American settlers in Callaway County. His parents were Thomas Harrison and Catherine (Maddox) Harrison. Thomas Harrison was born in Virginia, and moved with the family to Missouri when he was less than a year old; he served as a volunteer during the war with Mexico; tried mining in Michigan for several years in the Thirties; went to California in 1849, but returned to Auxvasse in a few years and passed the remainder of his life in Callaway County, where he became a prosperous farmer and a man of wide influence in the community, exhibiting those qualities of helpfulness and thought of others so strongly marked in his son Charles.

Mr. Harrison's early education was acquired in the public schools, from which, in 1876, he entered the University of Missouri, receiving the degree of T. E. in 1879 and the degree of C. E. in 1880. During the first two years of his course he was a working student, his father refusing to help him until, as he expressed it, he found out "what the boy was made of" and what he wanted to do. In college he exhibited those qualities for thoroughness, ready debate, and argument, which were so marked throughout his life.

After being graduated from the University, he entered the service of the Mississippi River Commission as a Rodman, and assisted in measuring the discharge of the Mississippi at Fulton, Tenn., until August, 1880, when he was transferred to Grafton, Ill., with the grade of Recorder, continuing in the work of discharge measurements until October, 1881. He was then transferred to Lake Providence, La., as Assistant on hydrographical and topographical surveys. He was obliged to give up this work in June, 1882, on account of malaria, but was offered employment under the Corps of Engineers of the Army, and from June to December, 1882, was Transitman on a topographical survey of the Missouri River from Fort Pierre to Fort Randall, Dak.; from December, 1882, to September, 1883, he was Instrumentman on a topographical survey of the Red River of the North.

The construction of reservoirs at the head-waters of the Mississippi was then in progress, under the direction of the Army Engineers, and Mr. Harrison was engaged as Assistant in the construction of a dam across that river at Pokegama Falls, Minn., from September,

* Memoir prepared by Alfred Noble, Past-President, Am. Soc. C. E., and J. Waldo Smith, M. Am. Soc. C. E.

1883, to July, 1884, and as Engineer in charge of the completion of a dam at the outlet of Lake Winnebigoshish, Minn., during the remainder of the year. During the five years ensuing he was with an uncle in Chicago, in the live stock commission business.

From July, 1890, to May, 1891, Mr. Harrison took part as Assistant Engineer in hydrographical surveys and investigations for the Sanitary District of Chicago; he then went to Northern Michigan as Engineer in charge of the improvement of Carp River and of the construction of a sewerage system for the City of Ishpeming, returning in February, 1892, to the Sanitary District to take charge of surveys for the location of the Chicago Drainage Canal, and from August, 1892, to August, 1897, was Division Engineer in charge of construction of Sections 5 to 15, embracing excavation of various kinds on a large scale and much concrete and structural work, including, with other important features, the regulating works at Lockport, Ill. While with the Sanitary District, Mr. Harrison served under Lyman E. Cooley and Isham Randolph, Members, Am. Soc. C. E., successively Chief Engineers. This was the first opportunity afforded to show to the Engineering Profession his ability as an engineer and an executive, and his work gave him a high standing.

In the summer of 1897, the United States Board of Engineers on Deep Waterways was organized to make surveys and estimates for a deep waterway from the Great Lakes to tide-water, and Mr. Harrison was the first engineer engaged. As Division Engineer he had charge of two sections, one being the Niagara Ship Canal route, from Lake Erie to Lake Ontario, the other extending from Troy to Lake Champlain. On the completion of this work at the end of 1900, he went to Panama for the Isthmian Canal Commission, where he made surveys to verify the work of the engineers of the French Company and also made borings at the site of the Bohio Dam and at other points. From June, 1900, to May, 1902, he was Chief Engineer for the Denver Union Water Company, and designed and built the Lake Cheesman Dam, notable as sustaining a greater head of water than any other structure then existing, and without any leakage.

At the end of May, 1902, Mr. Harrison accepted the position of Principal Assistant Engineer of the East River Division of the New York Terminal of the Pennsylvania Railroad. The division included four tunnels under the East River, built in compressed air, this class of work continuing nearly three years. In April, 1908, he suffered an injury of the heart from too long a stay under pressure, from which he never recovered. The work was then so nearly completed that if he could have endured for another day, he would not have incurred any further risk. He attempted to resume professional work in April, 1909, as Deputy Chief Engineer for the Board of Water Supply of the City of New York, but after a year's trial was compelled to give it up, and from that time limited himself to consultations. He passed

the winter of 1910-11 in Southern California; in the winter of 1911-12 he was in Havana and was greatly interested in the raising of the battleship *Maine*, then in progress. In March, 1912, he visited the Panama Canal.

In the latter part of August, 1912, he contracted a slight cold which did not take a serious form until two or three days before his death. He was buried in the family plot in the churchyard of the Presbyterian Church at Auxvasse.

Mr. Harrison's engineering experience embraced a wide range of successful work. To every new problem he gave the most thorough study, from the first step to the completion, and his judgment thus founded and trained was seldom or never erroneous. During his many years of participation in important work, he trained and developed a large number of young engineers from whom he required the same thoughtfulness and thoroughness, and nothing was more certain to incur his criticism than lack of care or misdirected energy, but it was given so kindly that no bitterness followed, and he possessed the friendship and loyalty of his assistants to a remarkable degree.

His work at three important industrial centers—Chicago, Denver, and New York—had made him well known throughout the United States as a capable engineer, an executive of rare ability, and as a man of the highest integrity. That his further career would have been most distinguished, if his health had permitted, is evidenced by the large number of engagements he was obliged to decline.

He loved the truth in all things, and hated any form of deceit or misrepresentation; he was generous to a fault, but for the hypocrite or professionally impecunious he had no consideration. His patience and perseverance, as well as his thoughtfulness for, and kindliness to, those associated with him, was particularly marked, and he was always a most loyal friend.

His recreation and hobby was a most unusual one, in that he was the only engineer, or one among a few, who owned a racing stable. His stable, although small, was chosen with the same good judgment and attention to detail shown in his engineering work, so that his colors were often in the lead on the principal tracks of the country, and his percentage of winners was greater than in the stables of the wealthy owners.

Mr. Harrison was elected a Member of the American Society of Civil Engineers on March 2d, 1898, and was a Director from 1908 to 1910. He was a frequent contributor to the discussions on papers before the Society; in 1905, the Thomas Fitch Rowland Prize was awarded to him and S. H. Woodard, M. Am. Soc. C. E., as joint authors, for their paper entitled "Lake Cheesman Dam and Reservoir."*

He was also a member of the Western Society of Engineers, and of the University and Engineers' Clubs of New York City.

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 89.

HORACE EBENEZER HORTON, M. Am. Soc. C. E.*

DIED JULY 29TH, 1912.

Horace Ebenezer Horton was born near Norway, Herkimer County, N. Y., on December 20th, 1843. Four years before the Civil War, when he was 13 years of age, his parents moved from Norway, N. Y., to Rochester, a small town in the southeastern part of Minnesota, at that time a frontier country. Here he was a pioneer, at an age susceptible to all the moving influences of a new country. Consideration of his after life leads to the belief that, as a western boy, he acquired those qualities of thoughtful planning, of initiative, and of self-reliance, for which he was distinguished in his manhood. With a fine mind, a splendid physique, and a natural fund of energy, this pioneer life was a good foundation for a working life characterized by ability, activity, and usefulness.

Mr. Horton's education in the schools was limited, as the schools were limited. At that time it may be said that engineering was not even a Profession, and engineering schools were almost unknown. At best, only an elementary engineering education could be offered in the schools of this country. Mr. Horton, doubtless, enjoyed such school training as was then obtainable. As a boy he attended school in Rochester, and later, for two years, at Fairfield Seminary in Herkimer County, N. Y., and was probably 19 or 20 years old at the completion of his course in that institution.

Considering the schools which he attended, and the proficiency which he displayed afterward in his Profession, it is fair to designate him as a self-educated man, and, like all true engineers, he continued to educate himself to the end of his life.

The timber bridge, at Oronoco, Minn., of 186 ft. span, 60 ft. above the water, was designed and built in 1866-67 by Mr. Horton, at the age of twenty-three. That a mere youth should combine the technical skill, the address to convince the authorities that the design was adequate, procure financial assistance, and have the business capacity to execute such a work, point out a man of very unusual powers and qualities.

One must remember that in 1866 Minnesota was a remote part of the world; that "Trautwine's Civil Engineer's Pocket-Book" was not published until six years later, and that the mass of technical literature we now have was not then dreamt of.

The fact recalls Smiles' "Lives of the Engineers," those wonderful men, with little education, nearly all coming from the strip of north country across England, who gave us The Highway, The Canal, The

* Memoir prepared by the following committee: Onward Bates, Past-President, Am. Soc. C. E., Hugh L. Cooper, and Arthur J. Mason, Members, Am. Soc. C. E.

Railroad, and The Extended Use of the Steam Engine in Modern Industry.

It is probably not a mere coincidence that the first Horton to land in America, in the seventeenth century, came also from Lancashire. One can readily believe that, running through the seven generations from the first Horton settled on Long Island, the constructive instinct persisted.

Horace E. Horton's father, a carpenter and builder, seventy years ago, in his wanderings between New York State and the far West, happened on some town in Ohio where a church had to be moved. The job frightened local talent, but Horton, Senior, undertook the work, succeeded, and, in his own words, "It paid handsome."

Here we have the Engineer's instinct strong in Horace E. Horton; a physical difficulty becomes a challenge or dare to overcome it.

Mr. Horton's career is of chief interest to us because he was one of the first group of engineers, fast disappearing, who grew up with the art; whose feeling for all kinds of material, structures, action, was the result of contact with them, and, therefore, became incorporated with his five senses, and instinctive.

A generation later, the Engineer's approach to knowledge in his work had become a literary one.

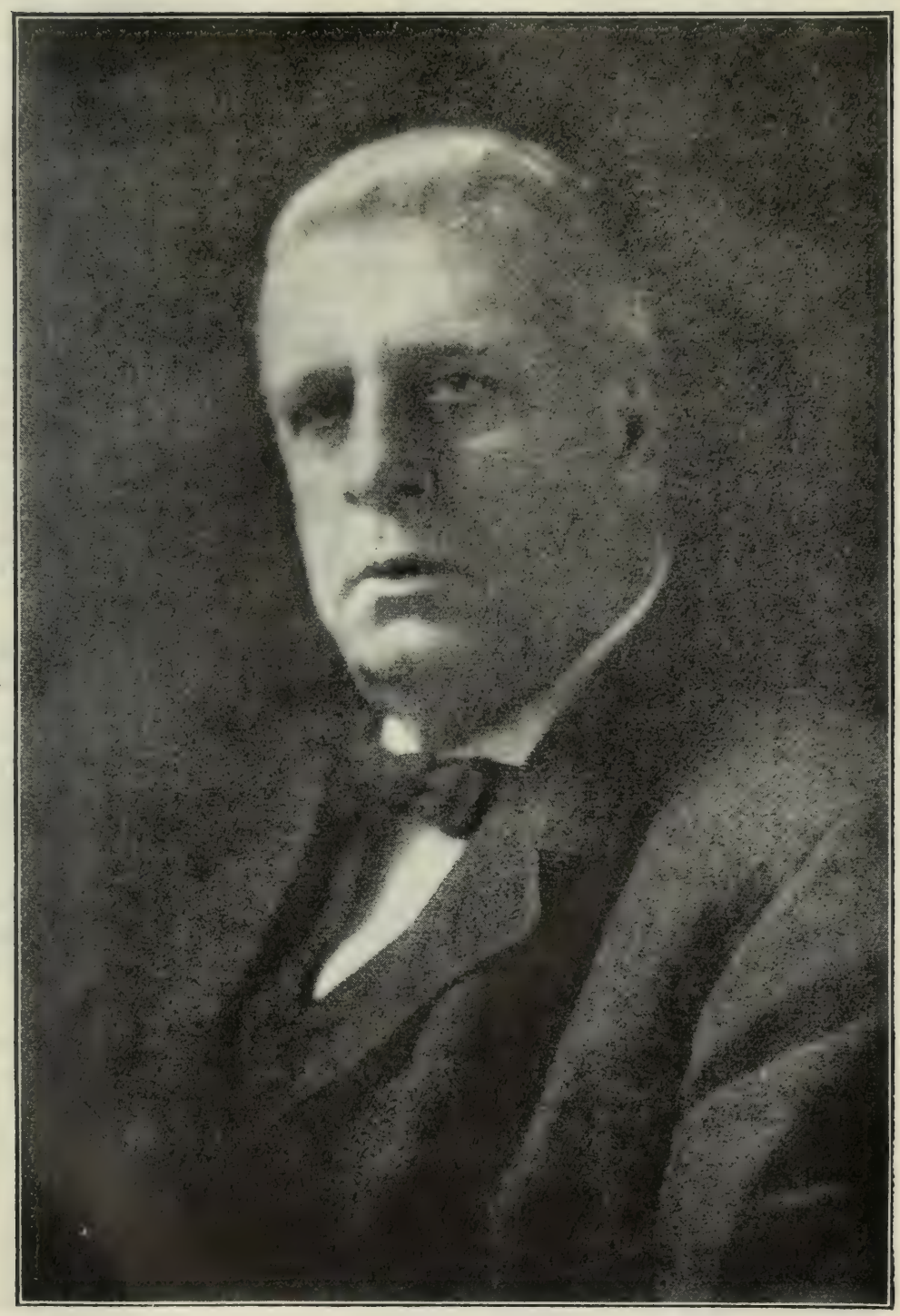
Horace E. Horton began his work with timber—the material of the country—and changed as the times changed. At his death his work was wholly in steel. He was always youthful in view point, always learning, never losing a sort of strong horse-sense.

Older members of the Society will recall his great impressive head, towering above the common run of men in any gathering.

Engineers are proud of the fact that the constructive quality seems ever to be accompanied with a certain robust force of character and probity. George Stephenson, the father of railways, a man who taught himself to read at the age of nineteen, was at home in the House of Lords, or any company, by reason of this mixture of sagacity, directness, and probity. Mr. Choate, in his address at the opening of the present Society House, traced these qualities to the fact that service and performance are the sole tests of the Engineer's work—not an appeal to taste, as so largely in other professions. From Horace E. Horton there seemed at all times to emanate an atmosphere of these very qualities—sagacity, directness, and probity. To this fact is due his success in the world.

Mr. Cooper joined Mr. Horton's staff in boyhood, and remained in his service for many years. He describes one side of the man as follows:

"Those who knew Mr. Horton intimately, and as an employer, realized his possession of a mind of unusual endowments, and specially gifted in creative analysis and quick comprehension.



HORACE E. HORTON.

"The subject might be engineering, which he dearly loved as a Profession, or it might be politics, history, genealogy, the Bible, or civic life. The rapidity of his understanding and general comprehension sometimes naturally led to little expressions of impatience with those of lesser equipment.

"One of his striking qualities in his engineering practice was his universal refusal in middle life to accept anything he had previously designed as a standard for the future. Every new contract for a bridge, and they were many, required an entirely new set of drawings to bring out new ideas of main and detailed designs, and in this sort of work he seemed to find great pleasure. The fact that in his later life he originated and perfected to a fine success the water-tank with the hemispherical bottom, that is now so generally used, and that he maintained this design as a standard over a term of years, during which time he built some twenty-four hundred of these tanks, was a great surprise to those who knew him between 1890 and 1900.

"He was a man who possessed the courage of his convictions to a striking degree, and having once settled a problem, either as related to a design or a method of constructing it, his mind seemed absolutely at rest and to forget the element of responsibility as usually defined. Precedents in methods of construction were tossed aside whenever opportunity for original and bold treatment was presented, and his really interested originality always came through to a successful conclusion.

"In general, his manner toward his employees was given to brusqueness, but they soon came to understand that this attitude was a shield for a nature overly kind when properly appealed to.

"One of Mr. Horton's personal characteristics, that was very rare among men of his ability and always in evidence with him, was his great patience with a student employee who was trying to better his condition through special studies after hours and through special hard work during working hours. To such he was never known to deny all the time the student asked for, and it frequently resulted in the student getting more time given him than he in his unwisdom thought he needed. His pride in the advancement of his 'young men,' as he called them, never came to the attention of the student, except when stories of their progress were told by Mr. Horton to third parties.

"His notions of charity followed most strictly the biblical inhibition about giving. He gave generously and freely to the needy, but always in a manner that successfully precluded the possibility of even minute public knowledge. During his residence in Chicago he listened to a very strong appeal from the pulpit for funds with which to purchase a pipe organ for the church. A day or two afterward he met the pastor and told him that he would give a certain sum toward the purchase of the organ, which amount happened to be 75% of the total cost, provided he would never breathe a word about the contribution to a soul. The pastor took the subscription, gave the exacted promise, and then went to another member of Mr. Horton's immediate family, stated that he had secured 75% of the necessary amount, asked for further aid, received 15% of the necessary amount, and gave the same promise about secrecy. The pastor knew from previous experience how to proceed, and was a skilled solicitor; at the first oppor-

tunity he appealed to yet another member of Mr. Horton's family, and obtained the remaining 10% for the organ, on the execution of the same promise regarding secrecy. It is believed that the family do not to this day know how that church got that organ, but the truthfulness of this story is vouched for, and is only related to evidence a trait of character beautiful and none too common.

"The possessor of great physical and mental strength, it is difficult to realize that he could so quickly succumb, and his going away will long be foundation for sadness in the minds of those he befriended."

As a bridge builder, his contracts were obtained by competitive bids, usually on plans prepared under his direction. These plans were required to conform to specifications and meet the approval of the engineer representing the purchaser. The successful bridge contractor had to design a structure meeting all requirements, and be the lowest bidder, as well. Success depended on the skill of the designer in producing the most economical structure to manufacture and erect. It was the custom for bidders to assemble at the time and place set for letting the contract, and the occasion was one in which competitors met in a friendly rivalry, although sometimes it was not so friendly. Competition was very keen, and, if by any chance a bidder lost his opportunity to bid, he was rather the butt of the jokes of the other bidders than an object of sympathy with them.

Mr. Horton was always a generous rival. He would encourage competition instead of suppressing it. It seemed as if he entered into the spirit of the game, on the principle that "the more the merrier," and that, if he won, his success was greater with many than with few to bid against. An instance of this is related by Mr. Bates, on the occasion of the letting of the superstructure for the high bridge at St. Paul. The bridge contracting company which Bates represented had arranged that the bidder's bond required by the city should be furnished to him through a local bank in St. Paul. On the morning of the letting of the contract, and perhaps two hours before the bids were to be opened, he called at the bank with full assurance that the bond would be passed over the counter to him. To his surprise and dismay, he was informed that some hitch, in the arrangement with the Philadelphia bank which was to guarantee the bond, prevented the St. Paul Bank from furnishing it. The decision was final, as it was impossible to make other arrangements in the limited time. Coming out of the bank, Bates paused on the steps of the building, reflecting on his position and wondering if anything could be done to save his bid. Just then Mr. Horton appeared, and, looking up, asked, "What is the matter? Why are you looking so sorrowful this morning?" He was informed of the dilemma of his competitor. Then Horton's fine instinct came to the rescue. He took Bates by the arm and said: "This will never do. We must get you a bond at once." Going to the nearest office build-

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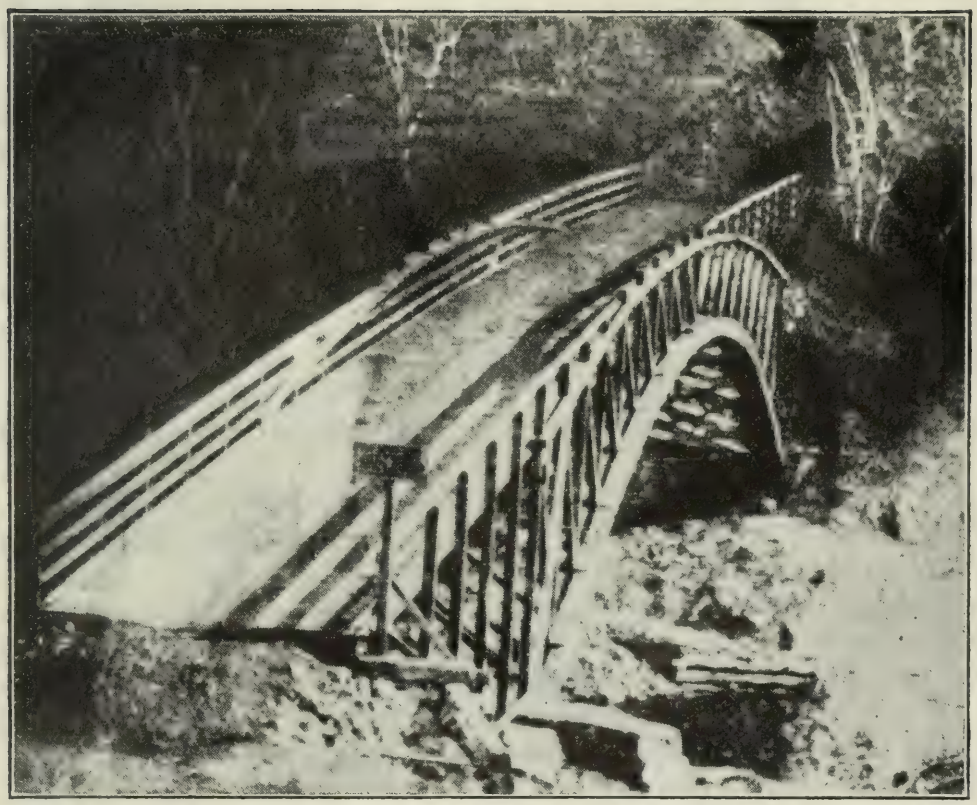

TIMBER BRIDGE, ORONOCO, MINN., 1866-67.

ing, they scanned the list of tenants, found the name of a lawyer and proceeded to his office. The lawyer was instructed to draw up the bond at once. Horton remarked: "The amount of this bond is so large that I cannot qualify for the whole of it myself, and we must get another bondsman." They proceeded from the building and on reaching the street, the first man who passed was seized by Horton and told that he must come in at once and sign Bates' bond, not even giving the newcomer an opportunity for argument. The matter was satisfactorily concluded. Bates' bid was considered, and when the decision on the bids was made known, the contract was Horton's as the lowest bidder.

Mr. Bates states that he never was so pleased at the success of a rival, and that a man who could show the spirit that Mr. Horton did under the conditions, was worth his lasting friendship.

Returning to Rochester, Minn., from his studies at Fairfield Academy, about twenty years of age, the next few years were devoted to whatever seemed to offer—farm work, surveying, railroad location—until we find him engaged on the Oronoco Bridge, described at the opening of this memoir.

With this as his starting-point, he was duly launched on his life work. For the following twenty-three years he had his headquarters at Rochester, and built up an extended business in the designing and erection of highway bridges throughout a considerable area of the Northwest. During this time he also developed some skill as an architect, designing a number of public buildings, as well as private residences.

A paper* entitled "Bridge Legislation," by Mr. Horton, is a delicious sample of the mixture of wit, clear-headedness, technical skill, and force of character of the man. Members will recall an occasion, years ago, when an effort was made to obtain acts of State Legislatures providing that highway bridges in the country should be under the supervision of some State officer or engineer, much as buildings are under the superintendent of buildings in cities.

Mr. Horton believed that this movement implied a failure to understand and appreciate the very remarkable ingenuity, skill, energy, and economy which had been shown by the men building highway bridges in the West, with funds seeming to us now humorously insufficient, in places remote, and under circumstances of extraordinary variety and difficulty.

Mr. Horton states a case which we venture to insert as a clean-cut sample of his logic and astonishing economy of design and execution.

"A congressional township (six miles square) with an assessed valuation of $38,000, had a stream running through it requiring a 200-ft. bridge; at medium low water the stream could be forded, but with a rise of one foot it was dangerous.

*Journal, Association of Engineering Societies, Vol. XI, p. 54 (February, 1892)

"Within ten years preceding the building of the bridge, three people had been drowned (one each, on three separate occasions) in attempting to ford the stream.

"With all available funds, and all ability for contracting a debt, the township was able to apply $1,800 for this work. It seemed to be best to build the bridge in one span.

"The abutments cost $500. A 200-ft. span was built for $1,300 (with profit to the builder), which has served eight years, with every prospect of use for a long term of years in the future.

"The thought comes to me: What would the engineers of legislation have done with a case like this. Would they have insisted on this community continuing to take chances on the ford, or have taken themselves out of the way, and allowed a man, with knowledge of how to do it, to build a bridge?"

Again, he says in the same paper:

"Have the committee been careless in their statements or shall I (having built thousands of highway bridges, and never having occasion to apologize for a failure either in design or execution) thank God for his mercy (being myself incompetent) * * * that these works have developed into bridges instead of hetchels."

Like most strong men, sure of themselves, he loved a "scrap."

Up to 1889 he had conducted his business, as a bridge contractor, independently of any shop. He had begun in the days of timber bridges, had passed through the combination stage, and had seen the metal bridge become the universal one. In 1889, recognizing that he must either become a manufacturer or lose his identity in the business, he moved to Chicago, and in conjunction with others started the Chicago Bridge and Iron Company, of which he was the President and Manager until the destruction of the plant by fire in 1897. At that time Mr. Horton purchased all the outside stock in the company, rebuilt the plant, and operated it thereafter in his own name, as the Proprietor of the Chicago Bridge and Iron Works.

Mr. Horton's great ability was as an engineer and as a contractor, rather than as a manufacturer. The interminable detail work of the latter annoyed him, and it was a great pleasure to him in later years to turn over much of this to his eldest son.

As an engineer, Mr. Horton was always progressive. When the combination of wood and iron became economical, he was prepared to use them to produce rational designs therewith. When the virtue of cantilever constructions for certain conditions was pointed out, he was ready to make use of them. A man of the most thorough commonsense, he utilized this in the design of his structures, which were proportioned to the service they were expected to give, rather than to some letter of a specification. At a time, twenty-five years ago, when communicating bridges were in demand by cities located on the banks of the Upper Mississippi River, where the conditions necessitated long

and high structures, with only a very limited amount of money available, he designed and built bridges which were strictly adapted to their service and have continued all these years properly to take care of the business. They were very light, and relatively inexpensive, structures, but they showed the skill of the designer, and the adaptation of means to ends. There was no metal wasted; every pound was used to the best advantage.

Perhaps Mr. Horton was best known to many people through his improvements in elevated water-tank design and construction. In 1894 he brought out a design for a steel water-tank supported on steel columns, the tank having a hemispherical bottom, and the columns connecting directly to the sides of the tank. Of this, his son, George T. Horton, M. Am. Soc. C. E., writes:

"He brought out these two essential elements which made the steel tank commercial with not over a half hour's study. The more I think of this one thing, the more remarkable it seems."

Prior to this, though the steel tank on steel columns had been used quite extensively, its general adoption had been prevented by the expense incident to the method of construction. The tanks had generally been flat-bottomed, supported on the columns by heavy floor-beams and closely-spaced joists, though there had been some instances of tanks with conical bottoms, and also with segmental bottoms, both these forms necessitating materially more metal, as well as more expensive shop work, than did the full hemispherical bottom produced by Mr. Horton. The method he adopted, too, for dishing these bottom plates, was novel and inexpensive. The result of these improvements, as well as of the energy and skill with which the sale of the structures was promoted by him, has been the general adoption of this form of elevated tank, and examples thereof are to be found in all portions of America, and the world generally, many of which are the product of his shop.

Mr. Horton's skill and courage as a contractor fully equalled and perhaps surpassed his ability as an engineer. In his case, the engineer and contractor were one. Being satisfied that a certain thing was feasible, he had the courage and skill, both as an engineer and as a contractor, to carry it out. If the state of the art did not provide the tools and facilities to suit him, he promptly devised them. He had a great deal of ingenuity, amounting to the true inventive instinct, and his shops contain many evidences thereof. So far as known, however, he never patented any improvement he made.

Mr. Cooper vouches for the following, which is worth telling as characteristic of the man, and goes to show that the same person may be at once shrewd and a money maker, and withal a generous man:

"Along about 1891 a strong competitor of Horton's was forced into the hands of a receiver. I happened to be in the office the next morn-

ing after the failure, and Horton, seeing the notice of the failure in the morning's paper, ordered me to bring him his check book, and he thereupon drew a check for $5,000 to the order of the President of that company and sent it to him with a brief note, stating that he undoubtedly needed a little household money and asked him to accept the check with his best compliments. As far as I know, and I think I am correctly advised, this check was never repaid, and was never expected to be repaid by Horton."

The foregoing incident seems to his biographers to be especially typical of the man, who, throughout his life, was large minded enough to act outside and above conventionality. He was probably influenced in this case, strange as it may seem, by the very reason that he was neither the same kind of a man nor an intimate friend.

Mr. Horton was married in 1871 to Miss Emma Babcock, of Waupun, Wis. She survives him, with their five children, George T. Horton, Mrs. K. Koessler, and Horace B. Horton, of Chicago; Mrs. R. H. Murray, and Hiram T. Horton, of Greenville, Pa.

He died at his home in Chicago on July 29th, 1912, aged 69, and was buried at Rochester, Minn.

Mr. Horton had a long list of friends who admired his personal qualities and who recognized his professional talent by electing him as President of the Western Society of Engineers, and as a Director of the American Society of Civil Engineers.

Mr. Horton contributed to the *Transactions* of this Society discussions on the following subjects: "The Halsted Street Lift Bridge," "Elevated Railroads," "Safe Stresses in Steel Columns," and "Tests of Large Steel Columns." He was the author of the following papers in the *Journal* of the Association of Engineering Societies: "Bridge Legislation" and "The Compressive Member." Mr. Horton's "Address," as President of the Western Society of Engineers may be found in the first volume of the *Journal* of that society, to which he also contributed discussions on: "Specifications for Steel Railroad Bridges," "Northern Pacific Standard Bridge Plans," "Specifications for Steel Structures," "Modern Practice in Bridge Shop Work," and "Curious Lattice Trusses."

Each of these papers and discussions will be found interesting, even to the layman, from the ability shown to discuss the most complex questions, in simple language, and comparatively free from formulas.

Isham Randolph, M. Am. Soc. C. E., in a letter to the writers, but voices, we believe, the judgment of the Engineering fraternity:

"Often we come in contact with men of heroic stature, whose only claim to consideration is a physical appeal to the eye; from within the human effigy there comes forth to fellow-man nothing helpful, nothing to be admired.

"How different it was with Horace E. Horton. There was a man of commanding stature, manifestly of great physical strength, but with

no faint suggestion of brute force. His noble head towered above his broad shoulders, and his whole appearance was but 'the outward and visible sign' of a great personality, dominated by a bigness of mind and of heart that made him both a leader and friend of his fellow-man, and won for him not alone the esteem, but the love of those with whom he came in contact in the varied relations of a life lived on a broad plan of usefulness.

"These are my epitomized thoughts of a man whom I admired and whose friendship I valued."

Horace E. Horton was elected a Member of the American Society of Civil Engineers on September 6th, 1882, and served as a Director during 1907-08-09.

WILLIAM BELL WHITE HOWE, M. Am. Soc. C. E.*

DIED FEBRUARY 11TH, 1912.

William Bell White Howe was the eldest child of William Bell White Howe who was originally from Claremont, N. H., and who, after studying for the ministry, went to South Carolina in his early manhood and became Bishop of the Episcopal Diocese of that State.

William Bell White Howe, Jr., was born in Charleston, S. C., on November 2d, 1851. After graduation at Charleston College he applied himself to the Civil Engineering Profession, and, after some preliminary experience on railroad surveys in Texas, was employed as Assistant Engineer on the harbor improvements at Charleston under the United States Engineer Corps. From 1876 to 1879 he was a member of the firm of Simons and Howe, Civil Engineers and Architects, in Charleston, and was Architect of the William Enston Home, in that city, as well as Chief Engineer of the Charleston Bridge Company.

Mr. Howe then became associated, as Assistant Engineer and afterward as Chief Engineer, with the system of railroads extending from Charleston into Georgia, Florida, and Alabama, popularly known as "The Plant System." He was instrumental in the substitution of iron bridges for wooden ones throughout the system, and also in the planning and construction of terminal stations and docks at Charleston, S. C., and at Savannah and Brunswick, Ga. As this system of railways was extended in the several States above mentioned, he was Chief Engineer of construction as well.

His ability as an engineer was made evident in the design and construction of the terminal station at Jacksonville, Fla., and more especially in the great work accomplished at Port Tampa, where a deep-water harbor was established on a shoal nearly a mile from shore in Tampa Bay. This harbor and system of docks afforded the best facilities for ocean traffic on the entire west coast of the Gulf of Mexico, and was made memorable as the base of operations against Cuba during the Spanish-American War. As an auxiliary enterprise to "The Plant System", Mr. Howe converted the street railway system in Jacksonville, Fla., into an electric railway, and built a power-house. He also planned the enlargement of the commercial docks at Key West, Fla., and the ocean pier at Palm Beach. Later, he withdrew from the more active pursuit of his Profession, and resided at Spartanburg, S. C., where he interested himself in architectural work and in water-power projects.

* Memoir prepared by H. S. Haines, M. Am. Soc. C. E.

Mr. Howe was of that turn of mind which is particularly adapted to a career of civil engineering. With considerable general culture, he was a close student of every branch of the Profession in which he was engaged. His early training as an architect was of advantage in planning important terminal structures, and, in connection with the works at Port Tampa and the Jacksonville Street Railway, he became well qualified as an electrical engineer. His researches into the properties of cement were recognized in his appointment as a member of the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers in 1897.

Only as occasion called for it was Mr. Howe's innate ability made manifest, for a reserve due to modesty screened his attainments from the observation of those who did not know him well; but, by reason of his conscientious discharge of duty and his unvarying kindness of demeanor, his more intimate acquaintances became warmly attached to him; and no one appreciated his personal qualities more than the writer of this memorial, with whom he was closely associated for many years. In the spring of 1911, his failing health caused him to retire to his summer home at Flat Rock, N. C., where he died from cerebral hemorrhage on February 11th, 1912, in the sixty-second year of his age.

Mr. Howe married Miss Louisa Preston King, granddaughter of the Hon. Mitchell King, of the South Carolina bar, who survives him. He is also survived by an only son, Dr. William B. W. Howe, of Hendersonville, N. C.

Mr. Howe was elected a Member of the American Society of Civil Engineers on March 1st, 1893.

THOMAS MOORE JACKSON, M. Am. Soc. C. E.*

DIED FEBRUARY 3D, 1912.

Thomas Moore Jackson, son of James Madison and Caroline Moore Jackson, was born at Clarksburg, W. Va., on June 22d, 1852. He received his early education at the public schools and at the Northwest Academy, of Clarksburg, and at Bethany College. He afterward entered Washington and Lee University where he took a special course in civil and constructive engineering, and from which he was graduated in June, 1873, with high honors, with the degree of Civil Engineer.

From 1874 to 1875, Mr. Jackson served as Chief Engineer of the Middle Island Railroad, in West Virginia, and from 1875 to 1879, he was Chief Engineer of the Weston and West Fork Railroad. In 1879, he was appointed First Assistant Engineer of the Iron Valley and Morgantown Railroad, later becoming Chief Engineer.

Mr. Jackson resigned this position to accept that of Chief Engineer of the Tunnelton and Kingwood Railroad, in West Virginia, where he remained until 1881, when he was appointed Engineer in Charge of mines at Wilsonburg, Clarksburg, Flemington, and Gaston, W. Va. Mr. Jackson had been engaged in mining engineering in various parts of West Virginia and Pennsylvania since 1875, and in co-operation with I. C. White, State Geologist, he opened and developed many of the coal, oil, and gas territories in various sections of West Virginia, being one of the pioneers in this field.

In 1882, Mr. Jackson was appointed Chief Engineer of the West Virginia and Pennsylvania Railroad. He held this position until 1885, when he resigned. He was retained, however, by the Company as Consulting Engineer for many years.

In 1887, he was made Assistant Engineer of the Clarksburg Water-Works and was also Engineer in Charge of geological maps of West Virginia and Pennsylvania, working on the coal fields of these States. In 1888, he was engaged in building coke plants at Wilsonburg, Clements, and Clarksburg, W. Va., and, in 1889, he went to Morgantown, W. Va., as Chief Engineer of the water-works and natural gas plant at that place.

In 1889 the Chair of Civil and Mining Engineering was established at the West Virginia State University, and Mr. Jackson was placed at the head of that Department. He remained in this position until 1891, when, the School of Engineering having been firmly established, he resigned against the protests of the Board of Regents and the Faculty of the University, to take up the active practice of his Profession.

After leaving the University, Mr. Jackson served as Chief Engineer of several railroads, among which was the narrow-gauge road from

* Memoir prepared by the Secretary from papers on file at the Society House.

Clarksburg to Weston, afterward known as the West Virginia and Pittsburg Branch of the Baltimore and Ohio Railroad.

Being prominent in the development of the coal, oil, and gas industries of that section of West Virginia about Clarksburg, and realizing the importance of a more direct route to the Ohio River from the latter place for the transportation of these products, Mr. Jackson undertook and was directly responsible for the building of the West Virginia Short Line Railroad, from Clarksburg to New Martinsville, which was completed and opened to traffic in 1901. He was President of this road until its purchase by the Baltimore and Ohio Railroad.

In 1910, Mr. Jackson organized and was made President of a company to build the Clarksburg and Northern Railroad to extend from New Martinsville, by way of Middlebourne and Salem, to Clarksburg, W. Va. Construction work on this road was begun in 1912, and the grading completed between New Martinsville and Middlebourne. Mr. Jackson had just returned to Clarksburg from New Martinsville, where he had been on business in connection with the new road, when he was seized with a severe attack of heart trouble which caused his death on February 3d, 1912, after a short illness.

In 1884, he was married to Miss Emma Lewis, daughter of Judge and Mrs. C. S. Lewis, who, with one daughter, survives him.

Mr. Jackson was a courteous and cultured gentleman, friendly and companionable on all occasions, who was esteemed and admired by all who knew him. He was one of the most prominent and progressive citizens of Clarksburg and was well known among business and professional men throughout the State. Practically his entire life was spent in his home town, and he did much toward its development as an industrial center. His ability as a civil engineer and his great knowledge of geology enabled him to realize the importance of the vast gas, oil, and coal areas in and around his home section and also the necessity of developing them for the establishment of manufacturing industries. It was through his efforts that the Jackson Iron and Tinplate Mills, now the Phillips Sheet and Iron Mills, which is one of the most valuable industrial plants in West Virginia, was established in Clarksburg.

He was President of the Traders National Bank of Clarksburg until its merger with the People's Banking and Trust Company into the Union National Bank, and was one of the chief promoters of the Traders Hotel, the building of which gave the city an up-to-date hotel.

The degrees of Mining Engineer, Doctor of Science, and Civil Engineer had been conferred on him by Washington and Lee University and by the West Virginia State University, respectively, and, as a member of Governor Fleming's staff, he had received the honorary title of Colonel.

Mr. Jackson was elected a Member of the American Society of Civil Engineers on June 9th, 1891.

WILLIAM FREDERICK LOCKWOOD, M. Am. Soc. C. E.*

DIED AUGUST 22D, 1912.

William Frederick Lockwood was born in New York City on July 4th, 1867, and, after attending the public schools, learned the trade of steam fitter. He entered the Engineering Department of what is now the Suburban Branch of the Third Avenue Elevated Railway during its construction in August, 1888, and on its absorption by the Manhattan Railway Company, in 1891, was taken into the Chief Engineer's Office, serving successively as Draftsman and Assistant Engineer until September 16th, 1900, when he was appointed Road Engineer, the duties of which office were the maintenance of the elevated railway structures.

While working as a Draftsman, Mr. Lockwood realized the limitations of his early education, and attended Cooper Institute in his spare time, receiving the degree of Bachelor of Science in 1894.

He was appointed Engineer of Maintenance of Way of the Interborough Rapid Transit Company on October 1st, 1905. His continuous term of service on the elevated roads covered twenty-four years, or practically his entire working life. During this time the roads were being extended, and improvements were being adopted, thus making the conditions of maintenance very burdensome, on account of the deeper foundations of buildings and numerous sub-surface structures, such as subways, conduit lines, high-pressure water mains, etc., which required the reconstruction of numerous foundations for the railway and the constant shoring of the structure at various points.

The adoption of electric motive power, requiring the reinforcement of the structure, was in itself a great task, and the constantly increasing street traffic rendered all such work more and more difficult. It is easily seen that such conditions would tend to keep a man of Mr. Lockwood's conscientious temperament very busy.

During all this time he had the care of his mother's family as well as his own. Time for study and almost for recreation was practically denied him.

His personality was somewhat unique, in that it was so fine. Of clear intellect, inflexible integrity, tireless industry, great patience, and a charming presence, he was a power for influence and accomplishment.

Mr. Lockwood died after a short illness following an operation for appendicitis. He is survived by his widow and three children, Ruth, Blanche, and Constance, and four sisters.

Mr. Lockwood was elected a Member of the American Society of Civil Engineers on October 4th, 1910.

* Memoir prepared by George H. Pegram, M. Am. Soc. C. E.

EDWARD MOHUN, M. Am. Soc. C. E.*

DIED OCTOBER 23D, 1912.

Edward Mohun, was born at Chigwell, England, on September 3d, 1838. In 1863, he came to Victoria, B. C., Canada, and during 1863 and 1864 was engaged as Assistant to the Surveyor General.

From 1864 to 1871, he was employed mainly on official surveys: laying out roads, etc., extending from Comox to Sooke, Nicola, South Thompson, and Bonaparte Valleys; surveying the Lower Fraser from Chilliwack to its mouth, Burrard Inlet, Squamish River, etc.; exploring Queen Charlotte Islands south; and making surveys for a waterworks system for Victoria and for the Wellington and Departure Bay Railway.

In 1871 and 1872, Mr. Mohun was engaged as Divisional Engineer on the Eagle Pass and Yellowhead Pass surveys for the Canadian Pacific Railroad. He also drove the first stake and made the first survey for this road on the Pacific Coast. In 1873, he explored the north end of Vancouver Island, and in 1875-76 was engaged in Government Surveys on that island and on the Lower Fraser River.

In 1876, he received the appointment of Surveyor to the Dominion and Provincial Joint Indian Commission, serving as Chief of the Survey from 1877 to 1884.

In 1885 Mr. Mohun was engaged on the reclamation and diking of 7 000 acres on the Lower Fraser River. While on this work he invented the sluice-box which was adopted by the Government and has been in use for more than 15 years, effecting the saving of many thousands of dollars.

In 1886, he made a test of the woods of British Columbia for which he received his proudest possession, a certificate signed "Albert Edward" and a medal. These tests agreed with similar tests made by the United States Government some twenty years later, and are the basis on which all strains in bridges built by the British Columbia Government are supposed to be calculated.

In 1887, Mr. Mohun designed and constructed the first portion of the sewerage system of Vancouver, B. C., the second portion being built under his supervision in 1889 and 1890.

In a competition for the design of a sewerage system for Victoria, B. C., in 1890, he was awarded the premium over eight competitors from Eastern Canada, the United States, and England, and was appointed Chief Engineer on its construction. As a result of the installation of this system, the death rate of Victoria was reduced 25% in three years.

* Memoir prepared by the Secretary from information on file at the Society House.

From 1893 to 1895, Mr. Mohun was employed as Provincial Government Railway Inspector, and, in 1895, he was appointed Engineer in Charge of the Pitt Meadows Dikes.

In 1898, he made a report to the Government on the sanitary condition of Rossland, Trail, Nelson, Kaslo, Revelstoke, Kamloops, New Westminster, Cumberland, Wellington, and Nanaimo. In the same year he entered the office of the Engineer of Public Works, and, in 1908, was gazetted as First Assistant Engineer. During this time he was engaged in the design of various public works, and since 1896 he had also acted as Consulting Engineer to the Provincial Board of Health, to which, in 1892, he had reported on the "Bacterial Treatment of Sewage." His conclusions on this subject were practically confirmed a few years later by the British Royal Commission on Sewage Disposal.

Mr. Mohun was the author of many engineering papers and of tables which are in constant use on the Pacific Coast. He was a Member of the Canadian Society of Civil Engineers, a Gzowski Medallist, and was also at one time Member of the Council of the Society. He was also a Member of the Royal Sanitary Institute, and for fifteen years was a Justice of the Peace for the Province.

Mr. Mohun was elected a Member of the American Society of Civil Engineers on April 6th, 1892.

WILLIAM HASELTON PUFFER, M. Am. Soc. C. E.*

DIED MARCH 17TH, 1912.

William Haselton Puffer, the son of the Reverend John M. and Anna Haselton Puffer, was born on May 3d, 1861, in Essex, N. Y. He attended the village schools until his fifteenth year, when, on the death of his father, the family moved to Chelsea, Vt., where he took up the management of a farm and the support of the family.

Mr. Puffer was not able to resume his schooling until he was 22 years of age. He then attended the Bradford Academy and later the St. Johnsbury Academy where he was prepared for college. He entered Dartmouth College in 1887 and the Thayer School of Civil Engineering in 1890, and was graduated from the latter in 1892. His struggles to obtain an education took him past the usual age for graduation from an engineering school, but the time was not lost. His early experience with adversity strengthened his will, broadened his character, and supplied him with a stock of common sense and practical experience which was of great value in his engineering work.

After his graduation, Mr. Puffer was employed on general railroad work with the Atchison, Topeka and Santa Fé Railroad until late in 1893, when he became Assistant Engineer in charge of construction for the Mexico, Cuernavaca and Pacific Railroad. In the summer of 1895 he served as Engineer for Sooysmith and Company, in charge of the foundations for the Cathedral of St. John the Divine, New York City. He returned to railroad work and, until 1897, was in charge of a party on the relocation of center lines for the Boston and Maine Railroad.

In the summer of 1897 Mr. Puffer again entered the employ of Sooysmith and Company and was engaged on the construction of the foundations for the Empire Building in New York City. In the fall of 1897 he entered the service of Joseph W. Cody and Company, as Engineer on shoring and deep foundations. During 1898 he had charge of concrete work for the U. S. Government at Fort Hamilton, and for the next two years he was Chief Constructor for the New York Zoological Society.

In 1901 he was employed by John W. Ferguson as Superintendent on construction of the plant for the Alsen Portland Cement Company, at Alsen, N. Y. After 1902, he was engaged in many mining enterprises in Mexico. He was Manager of the La Tula Mining Company, at Guanajuato, Mexico, from 1906 to 1912, and one of the Directors of the Company from its incorporation. He was also Manager of the Guanajuato Amalgamated Gold Mines Company, in 1909 and 1910.

* Memoir prepared by Arthur W. French, M. Am. Soc. C. E.

While on his way to Carlsbad, N. Mex., Mr. Puffer was stricken with cerebro meningitis, and died, after an illness of two days, in Pecos, Tex., on March 17th, 1912.

In September, 1892, he was married to Helen R. Davis, of Troy, Vt., who, with three children, survives him.

Throughout his life Mr. Puffer impressed friends, employers, and employees with his sterling uprightness of character, his kindness, his ability and good judgment, and his loyalty to duty.

Mr. Puffer was elected a Member of the American Society of Civil Engineers on May 3d, 1905.

JAMES DIX SCHUYLER, M. Am. Soc. C. E.*

DIED SEPTEMBER 13TH, 1912.

James Dix Schuyler was born at Ithaca, N. Y., on May 11th, 1849. He was the son of Philip Church and Lucy M. (Dix) Schuyler.

He received his early education at Friends' College which he attended from 1863 to 1868. He owed his high attainments in his Profession, and his versatility, very largely to subsequent study, a broad range of reading, and personal experience. As a young man he joined a railroad surveying party in Colorado, where he was stationed from 1869 to 1873, and, once imbued with the spirit of the West, he kept on to California where he made his home.

Mr. Schuyler began the practice of his Profession as Assistant Engineer on the location and construction of the Kansas Pacific Railway, in Western Kansas and Colorado, and was Resident Engineer on the last division of the road, remaining on this work until its completion in 1870. He was then appointed Resident Engineer on the Denver and Rio Grande Railroad, from Colorado Springs to Denver, and made the first survey of Colorado Springs. On arriving in California, in 1873, he was made Division Engineer of the North Pacific Coast Railroad from Ross Valley to San Rafael. In 1874 he was appointed Chief Engineer of the Stockton and Ione Railroad, and on the financial collapse of that project, he worked temporarily as a writer for the *Stockton Daily Independent*. In 1877 he was made Chief Assistant State Engineer, in the State Engineering Department of California, and was placed in charge of the irrigation investigations being conducted by that Department in the Great Central Valley of California.

Mr. Schuyler remained with the State Engineering Department until 1882, when he resigned to accept the position of Chief Engineer and General Superintendent of the Sinaloa and Durango Railroad in Mexico. In 1884 he returned to California, and was engaged for a time as a Contractor in the construction of a section of the sea-wall on the water-front of San Francisco. During the following year he was placed in charge of raising and completing the Sweetwater Dam, and the work which he did there assured his standing in the Engineering Profession, his rise to prominence thereafter being rapid.

Mr. Schuyler was called into conference and was placed in charge of important work in various parts of the United States and in Mexico. From 1891 to 1895 he was Consulting Engineer to the Lake Hemet Water Company, and designed and supervised the construction of the

* Memoir prepared by Stephen E. Kieffer, C. E. Grunsky, and J. B. Lippincott, Members, Am. Soc. C. E.

Hemet Dam, one of the highest masonry dams in Western America. He was engaged on the construction of water-works at Portland, Ore., and at Denver, Colo., on the Bear River Canal, in Utah, on the Ogden, Utah, Water-Works, and on irrigation works at various places in the West. He was one of the Board of Consulting Engineers to pass on the feasibility of the Owens River water-supply project, for Los Angeles, Cal., and was appointed by President Roosevelt as a member of the second Commission of Consulting Engineers which passed on the feasibility of the Gatun Locks, the Gatun Dam, and related structures for the Panama Canal.

His activities as a Consulting Engineer extended across the ocean to Japan, and as far south as Brazil. He was Consulting Engineer to the Territorial Government of Hawaii on the construction of the Nuuanu Dam; to the Monterey Water-Works and Sewer Company, Limited, of Mexico; to the Kobe Syndicate on an extensive power project in Japan, involving the construction of a very high dam; to the Mexican Light and Power Company, Limited, on the building of four large dams for power development in Mexico; to the Vancouver Power Company, Limited, on the building of a dam at Coquitlam Lake; to the Arrowhead Reservoir Company; the American Beet Sugar Company; and the Sweetwater Water Company.

Mr. Schuyler not only did much in a constructive way that will endure, but he had the happy faculty of imparting, through his writings, the results of his work and experience to other members of his Profession. Among engineers he was recognized as being unusually resourceful, and as having an unusual grasp on the experience of other members of the Profession, which he understood how to apply.

In writing of Mr. Schuyler and his work, Dr. George Wharton James says:

"His unfailing good humor, ready acquiescence in every experience, cheerful bearing of hardship, inconvenience, and annoyance, his uniform kindness, urbanity and courtesy, his gentle consideration for his associates, his companions, and the employees and helpers with whom he came in contact, his ready and quick response to every human and humane feeling and sentiment, his kinship with every living thing, whether of animal or plant life, his keen powers of observation and far-reaching grasp of reflection, his tenacity of memory, and thorough conscientiousness in getting hold of every fact that would aid him in his decisions, his persistent questioning and careful recording of the results obtained, his joy in recognizing the possibilities of a great project, his ready acknowledgments of errors in judgment based upon imperfect knowledge, his eloquent and forceful presentation of facts as he saw them, the brilliant keenness of his intellect in dealing with knotty problems of his Profession, the far-seeing scope of his fine and trained imagination, his loyalty to his friends, his kind toleration of those who harshly differed from him, his open devotion, sincere

admiration, and genuine affection for his wife, his brothers, his sisters, his family generally—these and a score of other traits clearly manifest in our association endeared him to me in a manner peculiarly potent.

"Physically, he was a large man, a man of impressive appearance, yet he was larger and more impressive mentally and spiritually than he was physically. A man of the keenest spiritual perceptions, he was yet a man among men, without a trace of the egotist or prig. His smile was an outward expression of a kindly, genial, humane nature. He wore no celluloid smile, formed by facial control. When his face beamed upon you it was the spontaneous, natural, ready response to the smile of his soul, warming you to trust, confidence, belief. He called forth the best of every man with whom he came in contact. He was a mental and spiritual dynamo, generating and radiating activity, peace, and good will to men.

"In his Profession he ranked with the highest and best, and at his funeral in Ocean Park, in September, every prominent member of the engineering and cognate professions of Southern California was present in person or by representative, and the expressions of the sense of loss were intimate and personal. The world has too few such men; would there were more."

In July, 1889, Mr. Schuyler was married to Mrs. Mary Ingalls Tulliper, of San Diego, Cal., who survives him.

He was a member of the Institution of Civil Engineers of Great Britain, the American Geographical Society, the Franklin Institute, the Technical Society of the Pacific Coast, and the Union League. He contributed extensively to various technical and scientific journals and Society publications, his best-known work being his book entitled, "Reservoirs for Irrigation, Water Power, and Domestic Water Supply," which was first published in 1901, a second edition having been issued in 1908. He was twice winner of the Thomas Fitch Rowland Prize given by this Society for the best paper presented during the year, the first in 1889 for the paper "The Construction of the Sweetwater Dam,"* and the second in 1907 for his paper "Recent Practice in Hydraulic-Fill Dam Construction."†

Mr. Schuyler was elected a Member of the American Society of Civil Engineers on December 6th, 1882. He also served as Vice-President in 1903-04, and as a Director in 1892, and from 1899 to 1901.

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* *Transactions*, Am. Soc. C. E., Vol. XIX, p. 201.

† *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 196.

FRANK SOULÉ, M. Am. Soc. C. E.*

DIED FEBRUARY 14TH, 1913.

Frank Soulé was born at Woodville, Miss., on August 6th, 1815. He came from New England ancestry, being a descendant of the Mayflower Pilgrims, and tracing his lineage directly to John Alden, Priscilla Mullins, and George Soulé. His father, Frank Soulé, went to California in May, 1849, where, later, he was well known as a journalist and as one of the authors of the "Annals of San Francisco."

Professor Soulé received his early education at Bridge Hampton, N. Y., and at the San Francisco High School. In 1862 he was appointed to the United States Military Academy at West Point, from which he was graduated in 1866. Immediately thereafter, he was assigned to duty as Assistant Ordnance Officer. In 1868 he was ordered to West Point as Instructor in Ordnance and Gunnery, and also performed the duties of Assistant Professor of Mathematics.

In 1870 he resigned from the Army to join the Faculty of the University of California, as Assistant Professor of Mathematics. In 1872 the Board of Regents established the Department of Civil Engineering by creating a chair for that subject; and on the appointment of Professor Soulé to that chair, they added Astronomy to his responsibilities.

The records of the University of California for the period ending with 1880 will show how prominent was the work of Professor Soulé in those early days. He not only gave his time to Mathematics, Astronomy, and Engineering, but also organized the Military Department. In 1886 the Students' Astronomical Observatory was designed and built by him with special State appropriations. In the next ten years, under his direction and with the co-operation of the late Professor George Davidson, Hon. M. Am. Soc. C. E., many important geodetic studies were maintained and measurements made. He inaugurated the Weather Bureau Record, which later was distributed by the Department of Astronomy, and now is reported by the Department of Geography. In 1890, the exacting demands of the rapidly growing Civil Engineering College forced Professor Soulé to relinquish his interests in astronomical work, although he retained the title of Professor of Astronomy with that of Civil Engineering for several years afterward. In the early Nineties, after a visit to prominent Eastern and European schools, he built and equipped a testing laboratory which from a modest beginning has grown into the present Experiment Station of the Civil Engineering Department.

Professor Soulé continued in the active service of the University as Professor of Civil Engineering and as Dean of the College of

* Memoir prepared by C. Derleth, Jr., M. Am. Soc. C. E.

Civil Engineering until the summer of 1907, a period of 38 years. In 1907 he spent a sabbatical year in foreign travel, visiting Egypt and most of the countries of Europe. On his return to California in 1908, he was retired on the Carnegie Foundation, becoming Professor Emeritus of Civil Engineering. Although his active work had ceased, he still retained a deep interest in the welfare and rapid growth of the College which he had organized. It is given to few men to serve an institution for a span of 44 years and in such varied capacities.

He was a man of multifarious tasks and of many-sided usefulness. No one was in closer touch with the students than he, both by virtue of his one-time office as Commandant of the Cadets and by the nature of his own interests. He was a military man in bearing and in thought, and believed in discipline and in authority. He laid upon himself the heavy hand of self-control, but was always kindly toward and thoughtful of others, and, above all things, was a man of unfailing courtesy.

During the last ten years of his service, it was his deserved pleasure to witness the great expansion of the Engineering Colleges of the University. From 1870 to 1900 his school, like San Francisco, developed slowly. It was about 1895 that, as a University, California began to grow with leaps and bounds. In his immediate Civil Engineering College a Department of Irrigation was established; courses relating to Forestry and Public Health were planned; and the relations between Agriculture and Engineering were defined. Sanitary studies formed a vital part of the curriculum, and instructors in Municipal and Sanitary Engineering were called to co-operate with the Medical Department in Bacteriology, Animal Industry, and Hygiene. In Civil Engineering there is now close contact between the sanitary instructors and the interests of the University known as Domestic Science. The Professors of Sanitation and Irrigation are concerned with the practical Agricultural School at Davis. The Structural Departments are associated with Architecture, no less than with Engineering. The Testing Laboratory is in touch with State problems. Surveying courses lead to instruction in Geodesy, through which channel the students are brought in contact with the Department of Astronomy. Through railroad work they deal with Economics; by the writing of contracts and specifications they lean on the Law; in the design of buildings they become interested in fire-proof construction, fire protection, and prevention, and are led to commune with the Departments of Economics and Insurance. All these relationships now exist, are daily developing, and are bringing Civil Engineering into closer sympathy and bond with the University. Professor Soulé served his University for 44 years; he saw it start from the humblest beginning, and witnessed its growth into one of the most powerful institutions in the country, a University which, in 1913, is second in numbers in the United States. More-

over, as an Instructor in Engineering he lived through those eventful years in the development of Engineering Education—from the pioneer days of instruction in Applied Science to the present decade of great promise.

Professor Soulé was always interested in the public affairs of the community in which he lived. Whenever a movement looking to a betterment in politics and community life developed in California in recent years, that movement found in him a ready helper. In earlier days his active interest was in Oakland; later, this was transferred to Berkeley, where he was a member of the Committee which framed the now famous New Charter for that city. He was an active member of the City Club of Berkeley, and often presided and spoke at citizens' meetings.

As Professor Soulé gave much of his energy and life to community interests and problems, it was only natural that he should be associated with many societies. He was a member of the Association of Graduates of the U. S. Military Academy, and of the Loyal Legion, U. S. A.; Colonel in the National Guard; life member of the Geographical Society of the Pacific; member of the Society of Mayflower Descendants; and life member of the Society of California Pioneers. For many years he was a member of the Athenæan Club, of Oakland, and of the University Club and the Bohemian Club of San Francisco.

Professor Soulé did not practice continuously his Profession as a Designing and Consulting Engineer; he was too busy with educational and social tasks, as already stated. Nevertheless, for thirty years he was a force in Engineering circles in the State of California, often serving in an advisory capacity or as a Consulting Engineer. He was a judge of materials and their tests, and frequently reported on road, street, sewer, and water supply projects for Berkeley, Oakland, and other growing towns and cities. Perhaps his most important reports were those which related to irrigation studies in the San Joaquin Valley and to the difficult foundation problems for the Ferry Building and adjoining docks in San Francisco.

He is survived by his widow, Mrs. Adelaide S. Soulé, and two sons.

Professor Soulé was elected a Member of the American Society of Civil Engineers on March 1st, 1905.

EBENEZER SMITH WHEELER, M. Am. Soc. C. E.*

DIED JANUARY 5TH, 1913.

Ebenezer Smith Wheeler was born at Buckingham, Wayne Co., Pa., on August 27th, 1839. In 1844, he moved with his parents to Noble County, Indiana, and it was there that he received his early training preparatory to entering the University of Michigan. He was graduated from that University in the Class of 1867, with the degree of Civil Engineer. Later, in 1897, on account of his many engineering achievements and general culture, his Alma Mater conferred on him the honorary degree of Master of Science.

Mr. Wheeler's entire engineering work was performed in the service of the United States, and mainly in connection with projects for the improvement of navigation on the Great Lakes. He has left many enduring monuments which will continue to make navigation safer and reduce the cost of transportation; these bear testimony more clearly than words as to his engineering ability. He first entered the service of the United States under the U. S. Lake Survey in 1863 as Assistant Engineer, and continued in that work until 1882. While in this position, Mr. Wheeler had much to do with the early triangulation of the Great Lakes, and to the accuracy of his work is due largely the reliability of the charts and maps used as guides by navigators. To his care was entrusted the delicate work of determining the lengths of the primary base lines in the Great Lakes triangulation system, on which all extensions of this system were founded. While engaged on this work, Mr. Wheeler devised the method of using a metal bar in melting ice as a means of standardizing steel tapes and base-measuring apparatus. This method of standardization was afterward adopted and generally used by others in this branch of engineering.

In 1882, Mr. Wheeler entered the service of the U. S. Engineer Corps under the officer stationed at Detroit, Mich. From that time until 1897, he was stationed at Sault Ste. Marie, Mich., in immediate charge of all Government work in that vicinity. Under his direct supervision were constructed the Poe Lock, the widening and deepening of the St. Marys Falls Canal, the Hay Lake Channel, the 20 to 21-ft. channel through St. Marys River, and many minor improvements to navigation.

In 1897, Mr. Wheeler was designated by the authorities at Washington to supervise the making of a preliminary survey for the Nicaragua Canal project in Central America. During 1897, he had direct charge of all field parties in Nicaragua, and during 1898, he was in Washington preparing his report and plans for submission to

* Memoir prepared by Charles Y. Dixon, M. Am. Soc. C. E.

the Nicaragua Canal Commission. Mr. Wheeler's report was so complete that there was no doubt that the Nicaragua Canal was a feasible engineering project, but this route was afterward abandoned for the Panama Canal. While engaged on his work in Nicaragua, Mr. Wheeler contracted a fever from the effects of which he never fully recovered, although, up to that time, his had been a most rigorous constitution.

On his return from work with the Nicaragua Canal Commission, in 1898, Mr. Wheeler was assigned to duty in the U. S. Engineer Office at Detroit, Mich., as Chief Assistant Engineer, and he continued in that position until his death. During this period, he had direct charge of the construction of the new St. Clair Flats Canal and of the breakwater at Mackinac Island. His engineering experience was also found to be invaluable when acting in an advisory way in connection with all other important works of the District.

While stationed at Detroit, Mr. Wheeler found time to make many investigations of value along lines indirectly connected with his work. The most important of these led to his invention of the Wheeler bathometer. This device has been placed on a number of freight vessels on the Great Lakes to give warning when in shoal water. It has been operated successfully for several seasons, and is destined to be the means of saving life and property on the Great Lakes.

A mere recital of the works with which Mr. Wheeler was connected is ample evidence of his engineering ability; but he was much more than an able engineer; he was well informed on many subjects, about which knowledge would be expected to be obtained by technical training only. He was, withal, a modest, unobtrusive, cultured gentleman. He was gifted with a keen sense of humor, and had that rare and happy faculty which enabled him to solve the difficulties of others and to illustrate his own thoughts with a good story well told. He was a most delightful companion, whom it was a genuine privilege to have known, and no one could know him well without being the better for it.

In 1874, Mr. Wheeler married Miss Clara P. Fuller who, with one daughter and one son, survives him. To his family the sympathy of a host of friends is extended, for truly all who knew him were his friends.

He was a member of many technical and fraternal societies, including the Michigan Engineering Society, the Detroit Engineering Society, the Detroit University Club, and the Masonic fraternity.

Mr. Wheeler was elected a Member of the American Society of Civil Engineers on November 7th, 1883.

HENRY FISHER WHITE, M. Am. Soc. C. E.*

DIED OCTOBER 28TH, 1912.

Henry Fisher White, son of J. Avery and Jane Elizabeth (Fisher) White, was born at Boston, Mass., on October 14th, 1851.

On the completion of his education, Mr. White, in 1870, entered the office of Messrs. Buttrick and Wheeler, at Worcester, Mass., which firm was engaged in general city and county practice. He also began his career as a railroad engineer about this time as Transitman on the Fitchburg Railroad.

In 1872, Mr. White was appointed Assistant to the Division Engineer of the Cairo and Vincennes Railroad on the construction of tunnels and heavy trestles on that road. He remained in this position until March, 1873, when he was employed as Transitman on a preliminary survey for a local company between Brookfield and Kansas City, Mo.

In June, 1873, Mr. White accepted the position of Division Engineer on the Ware Branch of the Boston and Albany Railroad, in charge of track-laying and ballasting on old grades. In 1874 and 1875, he was again engaged on general engineering work for Messrs. Buttrick and Wheeler, and on surveys for, and the construction of, minor roads. In September, 1875, he was employed as Transitman on location, and, in February, 1876, as Resident Engineer in charge of construction, of the Dayton and Southeastern Railway in Ohio.

In June, 1876, Mr. White was appointed Assistant to the Chief Engineer of the Burlington, Cedar Rapids and Northern Railway, with headquarters at Cedar Rapids, Iowa. In November, 1881, he was made Chief Engineer, which position he held until the road was made part of the Rock Island System. During his incumbency as Chief Engineer, the Burlington, Cedar Rapids and Northern Railway grew from 700 to 1 300 miles of splendidly built and equipped road.

When, in 1902, this road was absorbed by and became part of the Rock Island System, Mr. White was appointed Engineer of Maintenance of Way of the latter company, with headquarters at Chicago, Ill. He remained in this position until February, 1908, when he retired from active work. His later years were spent between his home in Chicago and his ranch near Montrose, Colo., where he died from an attack of heart failure.

On April 14th, 1880, Mr. White was married to Miss Anna McConnell, of Cedar Rapids, Iowa, who, with two daughters, survives him.

Mr. White was elected a Member of the American Society of Civil Engineers on January 2d, 1890. He was also a Member of the Western Society of Engineers and of the American Railway Engineering Association.

* Memoir prepared by the Secretary from information on file at the Society House.

CHARLES DE LA PLANE ATTERBURY, Assoc. M. Am. Soc. C. E.*

DIED JUNE 3D, 1912.

Charles De La Plane Atterbury was born at Howard City, Mich., on June 3d, 1878. His earlier years were spent in Utah, where he received instruction in private schools, and, for a short time, at the University of Utah.

He took up work as a telegraph operator at Murray, Utah, for the Rio Grande Western, and later was employed in the same capacity and as Agent on various roads and in various places in Oregon, Washington, and Idaho.

Mr. Atterbury began his technical education when he entered Leland Stanford, Jr., University, as a freshman in 1901. His sophomore year was spent at the University of Oregon, and his junior year at the Colorado School of Mines. Practical experience in mining engineering in Colorado was later found to be of considerable benefit to him in his chosen work. He again entered Stanford University as a senior in 1904, and it was here that the writer met and worked with him in various courses during the year, learning all the while the many sterling and admirable qualities of a character best known only to those in close association with him.

Before graduation, in the spring of 1905, he passed the examination for Engineering Aid in the Reclamation Service, with such standing as to merit immediate appointment. He was assigned to duty on the irrigation project at Hazen, Nev., for a short time on office work; later, he had charge of a party in the field. His ambition was not satisfied with the slower chances for advancement in Government work, so he resigned to enter the employ of a large corporation engaged in Alaska railroad development. On leaving this service he returned to Tacoma, Wash., and entered the employ of the Oregon-Washington Railroad as Division Engineer on a branch road near Centralia, Wash. After the completion of the road, he opened an office in Centralia, making a specialty of mine surveys in the coal region there, his previous experience fitting him for success in that work.

In the spring of 1908 Mr. Atterbury was appointed Assistant City Engineer of Centralia under Mr. Bullard, then City Engineer, and, on the resignation of the latter, some months later, was appointed to fill the vacancy. Many substantial improvements were made in Centralia and the neighboring city, Chehalis, during his term of office as City Engineer, for the success of which he was in large measure responsible.

A change in the form of government and personnel of city officers induced Mr. Atterbury to resign in the spring of 1912, dispose of his

*Memoir prepared by H. B. Langille, Esq.

home, and return to California. He immediately entered the employ of a large contracting firm in San Francisco, and was engaged in special construction, the requirements of which necessitated work in the tule bottom lands along the Sacramento River. During the excessively warm weather of 1912 his party suffered much from heat and lack of good water, Mr. Atterbury being much more affected than the others, due to his having come so recently from the cooler northern State. On the Friday preceding his death, which occurred on June 3d, 1912, he was partly overcome by the heat, but quickly recovered and made light of the occurrence. His wife, however, induced him to come to San Francisco for Saturday and Sunday, with the intention of giving up that work altogether. His characteristic conscientiousness and sense of duty to his employers' interests compelled him to return to the work, it being his intention to remain only Monday and Tuesday.

Work was prevented by the heat on Monday, but it was resumed on Tuesday, which, however, proved to be the hottest day of the year. In the tule rushes, which are 10 or 12 ft. high, the air does not move near the ground, hence is excessively moist and warm. The party started to return in the middle of the afternoon, two of the stronger men being sent ahead for water. On their return, while cautiously making their way through the tules to avoid being seen, they were observed by the others, and, as is usual in such cases when help arrives, Mr. Atterbury succumbed and fell. Some water and much rough handling brought him to his feet again, but for a short time only, when he fell once more and died before he could be carried to the nearest shelter. Two others of the party were overcome also, but were revived.

This was another instance of that quality of devotion to duty so frequently found in the Engineering Profession, and so strongly manifested in Mr. Atterbury. His was a character at once lovable, quiet, and unassuming; he was devoted to his family, loyal to his friends, purposeful, resourceful, and with high ideals of personality and profession; a clean, manly man whose passing is a distinct loss to his Profession and to the community in which he lived.

In May, 1907, Mr. Atterbury was married to Miss Ida Small, whom he met while at Stanford, and who, with two children, survives him.

Mr. Atterbury was elected an Associate Member of the American Society of Civil Engineers on November 8th, 1909.

ARTHUR GARFIELD CRYSLER, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 22D, 1912.

Arthur Garfield Crysler, the son of George and Harriet (Safford) Crysler, was born at Syracuse, N. Y., on April 24th, 1880. After going through the graded schools and the Syracuse High School, he entered Syracuse University in 1900, and was graduated in 1904, with the degree of C. E.

Immediately after his graduation, Mr. Crysler entered the service of New York State as Rodman in the Department of the State Engineer and Surveyor. He was assigned to work on the preliminary surveys, plans, and estimates of Contract No. 12, of the Barge Canal. In 1907 he was made Assistant Engineer and placed temporarily in charge of that portion of Contract No. 12 between Brewerton and Three Rivers, on the Oneida River, 12 miles of heavy work, including one lock. He did so well that the temporary appointment was made permanent, and he was in charge of the work at the time of his death.

On June 26th, 1906, Mr. Crysler was married to Miss Pearl LaShier, of Ithaca, N. Y., who, with three children, survives him.

Mr. Crysler was elected Associate Member of the American Society of Civil Engineers on August 31st, 1909. He was also a member of the Technology Club of Syracuse, N. Y.

*Memoir prepared by Guy Moulton, M. Am. Soc. C. E.

JOHN HAWKESWORTH, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 10TH, 1912.

John Hawkesworth, the son of James A. and Ella Zebley Hawkesworth, was born in New York City, on May 18th, 1883. He prepared for college at the Cutler School, and, in 1900, entered Columbia University, from which he was graduated in 1904, with the degree of C. E.

In May, 1904, Mr. Hawkesworth entered the employ of Raymond F. Almirall, M. Am. Soc. C. E., as Assistant Engineer, and was engaged on the design and superintendence of construction of various reinforced concrete buildings and structures in and around New York City. From 1906 to 1908, he served as Business Manager for Mr. Almirall and had general supervision of the work of the office.

Mr. Hawkesworth was one of the first engineers to engage in structural tests of reinforced concrete, and the results of his experiments were published in 1904 in "Cements, Mortars, and Concretes," by Myron S. Falk, M. Am. Soc. C. E. Other articles on this subject were written by Mr. Hawkesworth and published in *Engineering News* in 1905. He was also the author of a "Graphical Handbook for Reinforced Concrete Design," which was issued in 1906, and of various other technical books and papers, one of the latter being entitled "Precarious Expedients in Engineering Practice," presented before this Society on February 16th, 1910.†

From 1908 to 1912, Mr. Hawkesworth spent part of his time in travel abroad and part in consulting engineering practice in this country. He was also engaged in investigating corporations desirous of increasing their capital. Shortly before his death he organized The Carleton Company, Engineers and Contractors, of New York City, of which he was President and also a Director.

Mr. Hawkesworth was married, on November 25th, 1911, to Miss Marion Everett Barling, who survives him.

He was a member of the Lotus Club, the Columbia University Club, the Westchester Country Club, the Municipal Art Society, and of Psi Upsilon.

Mr. Hawkesworth was elected a Junior of the American Society of Civil Engineers on September 6th, 1904, and an Associate Member on November 4th, 1908.

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* Memoir prepared by Milton L. Cornell, Esq.

† *Transactions*, Am. Soc. C. E., Vol. LXVII, p. 32.

WILLIAM MADISON MYERS, Assoc. M. Am. Soc. C. E.*

DIED APRIL 4TH, 1912.

William Madison Myers, son of William H. and Mary Jane (Harman) Myers, was born near Winchester, Va., on April 4th, 1873. His parents, who lived on a fine farm in the Shenandoah Valley, perceiving that the son had a fondness for books and study, and realizing the value of education, determined to fit him for a professional life. After availing himself of all the advantages of the country schools, Mr. Myers entered the Winchester High School, from which he was graduated in 1891. In the fall of the same year he entered Washington and Lee University, and was graduated from that institution in 1895, with the degree of Civil Engineer.

For some time after his graduation, he taught in the Graded Schools of Frederick County, Virginia, being engaged occasionally on local surveying and engineering work. In 1897 and 1898, he was in charge of road improvements between Leesburg and Harper's Ferry, Va., under Frank Conrad, Civil Engineer.

In 1898, Mr. Myers accepted an offer from the late Lewis Kingman, M. Am. Soc. C. E., then Chief Engineer of the Mexican Central Railway, to go to Mexico as Transitman on the location of the Tampico Division of that railway, being also at times in charge of a party locating reservoir sites, etc. In 1900 he was appointed Assistant, under Mr. W. C. Harris, on the construction of the Parral Extension of the same road, and, in 1901 and 1902, he was with Mr. O. G. Bunsen, as Transitman and Topographical Draftsman, on the location of the Colima Extension.

From 1902 to 1904, Mr. Myers was employed as Division Engineer on the construction of the San Pedro Extension of the Mexican Central Railway, and, while in this position, had charge of the construction of 50 km. of the road, as well as of all the Division yards and buildings. He remained in this position until the work was completed, when he returned to Virginia to visit his old home. After seven years' service on the Mexican Central Railway, under Mr. Kingman, the latter wrote of him:

"Mr. Myers is a gentleman and an educated engineer, and has done all his work, while under my jurisdiction, with credit to himself and to the entire satisfaction of the Railway Company. I take pleasure in recommending Mr. Myers. He is honest, truthful, and considerate. He is industrious, temperate, and careful in his work. He understands the classification of material, and has always done the right thing with the contractors and the Railway Company."

* Memoir prepared by Mrs. Bessie B. McCann.

In 1904, Mr. Myers was appointed Transitman and Topographical Draftsman on the Florida East Coast Railway, but in less than a year, because of the climatic conditions, was compelled to resign this position and return to his home to recuperate.

In 1905, he went back to Mexico, as Division Engineer, under Mr. Kingman, on the construction of the Colima Extension of the Mexican Central Railway from Tuxpan to Colima. This work was very heavy and included the construction of three tunnels in a distance of 7 km. In 1906, Mr. Myers accepted a position with Pierson and Sons, of the Puebla Tramway, Light and Power Company, at Puebla, Mexico, his work consisting of city surveys, and surveys and estimates for water-works, hydro-electric plants, electric lines, etc. Of him and his work at this place, Mr. Pierson has written:

"I consider him an able and experienced railway and general engineer, competent to take charge of important work. * * * As to Mr. Myers' character, I have the highest opinion of him. He is of good family, sober, reliable, and conscious of the dignity of his Profession."

After spending three years in Mexico, Mr. Myers returned to his home in Virginia and was again engaged in local engineering around Winchester. In 1910, he entered the employ of the Virginia State Highway Commission, under Mr. P. St. J. Wilson, having been located at Buena Vista, Big Stone Gap, and Fairfax, Va. While engaged at the latter place, in January, 1912, he was taken ill with typhoid fever and removed to the Georgetown University Hospital, where he remained under treatment for two months. Failing to improve, he was removed to his home in Winchester, where he died on April 4th, 1912.

Mr. Myers was always a student, and surrounded himself with the latest works on all branches of engineering. He was painstaking, and all his work and papers show that great care and precision were his marked characteristics. He was a member of the Presbyterian Church, and carried his love of God and man in his work, always living his religion.

Mr. Myers was elected an Associate Member of the American Society of Civil Engineers on October 3d, 1906.

CHARLES HARRY TISDALE, Assoc. M. Am. Soc. C. E.*

DIED APRIL 30TH, 1913.

Charles Harry Tisdale, the youngest child of Benjamin F. and Eliza H. Tisdale, was born in New Orleans, La., on May 30th, 1874, but after living there only a short time the family removed to the country just outside of Baton Rouge, La., where his early boyhood was passed. He attended the small country schools, but learned principally from his mother's tuition. The family afterward removed into the City of Baton Rouge, where he attended the public schools. He then entered Louisiana State University, from which he was graduated in 1895 with the degree of B. S.

His first work was to teach school at Brusle' Landing, West Baton Rouge Parish, La., where he met the young lady who afterward became his wife.

He taught school only one term, and then accepted a position under Col. George McC. Derby, M. Am. Soc. C. E., who then had charge of the Fourth Mississippi River District, with headquarters in New Orleans. Mr. Tisdale was in that office only a short time, being employed in the Drafting Department; afterward he went into the field as Inspector of levee construction.

In 1902 he was transferred to the Mobile, Ala., District, where he worked in the Drafting Department for about a year. He was then sent into the field and served successively as Recorder, Inspector, and Junior Engineer, in connection with the building of locks and dams on the Alabama and Tombigbee Rivers, Ala.

In the early part of 1906, Col. H. C. Newcomer, who was then in charge of the Chattanooga, Tenn., District, sought a capable man to place in local charge of the projected lock and dam at Hales Bar on the Tennessee River. Mr. Tisdale was selected for the position, was appointed Junior Engineer on February 1st, 1906, and placed in charge of the work. He was there when ground was broken, and remained in charge until his death. There he had his greatest sorrow, his young wife being called away without a moment's warning. His only consolation seemed to be his work, in which he absorbed himself. He saw it growing toward completion after many difficulties had been overcome, and yet he was not spared to see it completed. He knew every inch of the construction and every man connected with it in any way. When finished it will be an enduring monument to him. As the mighty Tennessee rolls onward past the dam, between the mountains standing like sentinels over it, as the turbines whirl in the current and manu-

* Memoir prepared by Robert R. Tisdale, Esq.

facture light and power for distant Chattanooga, they will bear witness to the faithfulness of the one who wrought so earnestly and yet did not live to see the completion of the great project.

Faithful to friends, loving toward his family, and zealous in the performance of his duty, he is mourned by many.

Charles Harry Tisdale was elected an Associate Member of the American Society of Civil Engineers on October 31st, 1911.

JAMES HUGH WISE, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 16TH, 1912.

James Hugh Wise was born in Denver, Colo., on February 27th, 1880. As his father was engaged in mining, the young man spent the first ten years of his life in various parts of New Mexico; he was then brought to San Francisco, Cal. During the next five years he attended the schools of that city and Alameda, and then entered the California School of Mechanical Arts, in San Francisco, from which he was graduated in June, 1899. In that year he entered the University of California, and received his degree from the College of Mines in June, 1903.

During the following year Mr. Wise taught mathematics at the California School of Mechanical Arts, resigning in June, 1904, to enter the employ of the California Gas and Electric Corporation, now the Pacific Gas and Electric Company. His rise to responsibility was strikingly rapid. He continued with the Corporation until February, 1910, when he resigned to become a partner in F. G. Baum and Company, Consulting Engineers. In July, 1911, Mr. Wise again entered the employ of the Pacific Gas and Electric Company, this time as Assistant General Manager, which position he held at the time of his accidental and tragic death. He was on an inspection of the Los Angeles Aqueduct by automobile. While in the southern end of the San Joaquin Valley, it was found that the gasoline tank was leaking, and a stop for repairs was made at a garage. While assisting to empty the tank, it accidentally fell, and Mr. Wise was saturated with the liquid. The day was exceedingly hot, and the gasoline vapor was ignited by a torch burning in a distant part of the garage. Before the flames could be extinguished, Mr. Wise was burned so severely that he died in Oakland the following day.

In his relatively short career, he had held a number of distinctly responsible positions. Among his earlier works may be mentioned the reconstruction of the Centerville Plant of the Pacific Gas and Electric Company, which consisted of the enlargement of the conduits, the rebuilding of the power station, and the installation of a turbine operating a 5 000-kw. generator under a 600-ft. head. Later, he had charge of the erection of a 5 000-kw. generator at Colgate, and of similar work for the Deer Creek Plant of the Pacific Gas and Electric Company. In 1909, the Lake Arthur Dam was constructed under his supervision.

At the time of his death, Mr. Wise was in direct charge of the design and construction of a large hydro-electric power plant for

* Memoir prepared by J. D. Galloway, M. Am. Soc. C. E., and B. A. Etcheverry and H. C. Vensano, Associate Members, Am. Soc. C. E.

his Company. This work included a masonry dam, more than 300 ft. high, to store the waters of Lake Spalding. The conduits, pipe lines, buildings, machinery, and steel tower transmission line of the plant were proportioned to generate 40 000 kw.

Mr. Wise, on account of his kindly nature, was uniformly liked by all who met him. He had exquisite tact, most courteous manners, and, in a very high degree, the power to command loyal service and friendship from others. The writers, speaking for themselves and for many who knew him, remarked with pleasure his rapid rise to a responsible position, and they take this occasion to express their deep respect for the noble character of a friend whose death is a cause for profound sorrow and regret. He was a Mason, was unmarried, and is survived by his mother who lives in Berkeley, Cal.

Mr. Wise was elected an Associate Member of the American Society of Civil Engineers, on February 6th, 1907, and, at the time of his death, was Vice-President of the San Francisco Association of the Society.

STEPHEN HOLMAN, F. Am. Soc. C. E.*

DIED OCTOBER 13TH, 1912.

Stephen Holman was born at Royalston, Mass., on December 28th, 1820. He was the son of Stephen and Hannah (Fuller) Holman whose ancestors settled at Newburyport, Mass., early in the Eighteenth Century, several having served in the War of the Revolution.

On the death of his father in 1832, the boy went to live with his half-brother at Saugus, Mass. After attending school at Saugus Centre, he entered the Lynn Academy where he prepared for college. He was graduated from Williams College in the Class of 1840. Afterward, he studied law and was admitted to the bar, but never practiced that profession to any extent, preferring that of teaching. For a number of years he served as principal of high schools and academies at Winchester, N. H., Gardner, Athol, Phillipston, Fitchburg, and Holyoke, Mass.

While in the latter city Mr. Holman was employed as Paymaster at the Lyman Mills, and, in 1860, bought a controlling interest in the Holyoke Paper Company, which he managed with conspicuous ability. In 1865 he sold his interest in that Company and established the Holyoke Machine Company, and, later, a branch of the same company at Worcester, Mass. He also founded the Deane Steam Pump Company, of Holyoke, and became interested in a number of cotton mills and also in the American Telephone Company.

Mr. Holman was very successful in business, and this success was due in part to his power of initiation. He was the originator of many new methods of reducing business to a scientific basis, and was the first to establish the present methods of cost-keeping in paper mills. An example of his original methods was the establishment of a testing flume at the works of the Deane Steam Pump Company, at Holyoke, by which every water-wheel manufactured at the plant could be tested, as to head and power, under actual service conditions, thereby guaranteeing them to purchasers. This flume was also placed at the disposal of any manufacturer who cared to make use of it for testing.

Mr. Holman retained his physical and mental vigor until his death, keeping in close touch with his business interests. He was a student and an accomplished linguist, never losing touch with his college studies, especially that of the German language and literature of which he was particularly fond. At the age of ninety he gave a reading from Schiller's plays before a German association.

He was a brilliant conversationalist, and, having a wonderful range of general knowledge, could entertain others no matter what their

* Memoir prepared by the Secretary from information supplied by C. J. H. Woodbury, M. Am. Soc. C. E.

tastes and experiences in life had been. He was tenacious in his friendships, particularly with men, his cultured mind and strength of intellect making him a dominating figure in personal associations and in business life.

Fondness for Nature was one of Mr. Holman's strong characteristics, which was gratified by extensive travel in this country and in Europe. He had just returned from a European trip when he was seized with a sudden attack of neuralgia of the heart which caused his death at Swampscott, Mass.

On April 12th, 1853, Mr. Holman was married to Miss Hannah A. Richardson, of Lowell, Mass., who died in 1894. He is survived by a son and a daughter.

He was a Member of the Mt. Tom Lodge, A. F. and A. M., and of many other organizations. At the time of his death he was the oldest Alumnus of the Lynn Academy and of Williams College. He was also the Senior Mason and the Senior Member of the Massachusetts bar.

Mr. Holman was elected a Fellow of the American Society of Civil Engineers on June 29th, 1872.

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